

Comparison of European and American Standards for the preliminary design of an Oil Tank Foundation

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By

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Contents

1.	Introduction	1
1.1	Overview	1
1.2	Norm used.....	2
1.3	Structure of the thesis.....	3
2.	Methodologies	4
2.1	Load and Resistance Factor Design Method	4
2.1.1	Mean and Characteristic value.....	4
2.2	Limit State Design method	5
2.2.1	Design Values of action	5
2.2.2	Design Values of geotechnical parameter	6
2.2.3	Eurocode ULS.....	6
2.2.4	Eurocode SLS	9
2.2.5	Load Combination for ULS.....	10
2.3	Allowable Design Method	12
2.3.1	Factor of safety for Bearing Capacity	13
2.3.2	Load Combinations.....	13
2.4	Differences	14
2.5	GEO and STR approaches considerations	15
2.6	Summary.....	15
3.	Shallow Foundation: Ultimate Bearing Capacity	16
3.1	Failure Mechanisms	16
3.2	Undrained Case	17
3.3	Drained Case.....	19
3.3.1	Brinch Hansen equation	21
3.4	Undrained Case Analysis	23
3.5	Drained Case Analysis.....	24
3.6	Summary.....	25
4.	Case Studied, Foundation Typologies, Shell Design.....	26
4.1	Geology of the area	26
4.2	Case Studied	26
4.3	Subsoil stratification	27
4.4	Soil strength results.....	27
4.5	Foundation Types	29
4.6	Summary.....	29

4.7	Shell Design	30
4.7.1	Shell	30
4.7.2	Annular Bottom Plate	31
4.7.3	Bottom Plate.....	31
4.8	Summary.....	32
5.	Wind and Seismic action Determination	33
5.1	Eurocode Wind Load	33
5.2	API 650 Wind Load	35
5.3	Methods Overview	36
5.4	Seismic Action.....	38
5.5	Differences and Similarities.....	38
5.6	Centre of action of the effective lateral forces.	41
5.7	Overturning Moment	43
5.8	Acceleration Coefficient	43
5.8.1	Design Response spectrum Eurocode	44
5.8.2	Design Response Spectrum API 650.....	45
5.9	Seismic Foundation design API 650.....	46
5.10	Seismic Foundation design Eurocode.....	46
5.11	Freeboard	47
5.12	Comparison	48
5.12.1	Impulsive and Convective time H/D dependency	49
6.	Foundation Design	51
6.1	Load Combinations.....	51
6.2	Ground Improvement.....	51
6.2.1	Preload.....	52
6.2.2	Time of Consolidation.....	55
6.2.3	Vertical Drains	56
6.3	Foundation Sizing	58
6.3.1	Case Studied	59
6.4	Circular Footing Stress Assessment.....	61
6.5	Settlements	63
6.6	Steel Reinforcements	65
6.6.1	ACI 318 Load combinations	65
6.6.2	Twist moment.....	66
6.6.3	Hoop Tension.....	68

6.6.4	Steel Area API 650 and EN 1993 comparison	70
6.6.5	Undrained Bearing Capacity	71
6.7	Drained Bearing Capacity	73
7.	Evaluation & Results Discussion.....	75
7.1	European and American Codes	75
7.2	Method used	76
7.3	Wind and Seismic Load.....	76
7.4	Load Combinations.....	77
7.5	Eurocode approaches.....	78
7.5.1	Approach Comparison Foundation Sizing	79
7.5.2	Approach comparison General Failure mechanism	79
7.6	Steel Reinforcement.....	80
8.	Recommendation.....	81
9.	Conclusions	82
	Appendix A Soil Bearing Capacity Equation	1
A.1	Undrained Case	1
A.2	Drained Case	2
	Appendix B Foundation Typology	B-1
	Appendix C Soil Test data.....	C-1
	Appendix D Wind load calculation	D-1
D.1	Wind Load EC1	D-1
D.1.1	Wind pressure.....	D-2
D.1.2	Wind Force.....	D-3
D.2	Wind Load ASCE 7-10.....	D-4
D.2.1	Wind Pressure.....	D-4
D.2.2	Wind Force.....	D-4
	Appendix E Seismic map hazard (Eurocode).....	E-1
	Appendix F Eurocode Combination Loads	F-1
F.1	Characteristic Loads	F-1
F.2	Design Loads [Above Foundation].....	F-4
	Appendix G API 650 Combination Loads.....	G-1
	Appendix H Load Combinations ACI 318.....	H-1
	Bibliography	1

List of Figures

Figure 1 "Probability density function distribution (Bashor, Kareem, & Moran, 2009)"	4
Figure 2 "Modes of bearing capacity failure: (a) general shear failure; (b) local shear failure; (c) punching shear failure" (Braja, 2009)	16
Figure 3 "Upper and Lower limit solutions Undrained Case" (Craig, 2004)	17
Figure 4 "Prandtl mechanism of failure (Craig, 2004)"	18
Figure 5 "Lower Limit solution Drained Case" (Craig, 2004)	19
Figure 6 "Terzaghi mechanism of failure" (Craig, 2004).....	20
Figure 7 "Mesopotamian Region Map"(Google Maps)	26
Figure 8 "Wind velocity pressure profile EN 1991-1-4:2004"	34
Figure 10 "Liquid-filled tank modelled: double single-degree-of-freedom system (Malhotra, Wenk, & Wieland, 2000)"	38
Figure 10 "Qualitative description of hydrodynamic pressure distribution on tank wall and base"	38
Figure 11 "Undrained Failure Mechanism referred to the case of Global Equilibrium (Duncan, ASCE, D'orazio, & ASCE, 1984)"	52
Figure 13 "Contact pressures and settlements for flexible foundation: (a) elastic material; (b) granular soil" (Braja, 2009)	53
Figure 13 "Contact pressure and settlements for rigid foundation: (a) elastic material; (b) granular soil" (Braja, 2009)	53
Figure 14 "Model scheme for stress distribution due to (a) uniform pressure and (b) linearly increasing pressure, " (Craig, 2004, p. 149)	54
Figure 15"Relationship between the coefficient of consolidation and the plasticity index for different effective vertical consolidation pressures (Sridharan & Nagaraj, 2012)"	56
Figure 16 "Ringwall loading scheme (PIP STE 03020)"	58
Figure 17 "Pressure bulbs for vertical uniform load, circular footing (Coduto, Yeung, & Kitch, 2011)"	61
Figure 18"Equilibrium scheme twisting moment (PIP 03020:2005)"	66
Figure 19 "Load distribution under seismic or wind load (PIP 03020:2005)"	67
Figure 20 "Hazard map, Middle East, Peak ground acceleration expected in 50-years period with a probability of 10%"	E-1

List of Tables

Table 1 "Design load partial factors, Approach 1"	8
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Table 2 "Design load partial factors, Approach 2".....	8
Table 3 "Design load partial factors, Approach 3"	9
Table 4 "Combinations load Factors, Ultimate Limit State (ULS) EN 1991-4:2003 table a.1"	11
Table 5 "Factors for seismic load combinations (ULS) EN 1991-4:2003"	11
Table 6 "Combination Load Cases according to Equation 2.14, Eurocode"	12
<i>Table 7 "Combination Load Cases according to Equation 2.15, Eurocode"</i>	12
<i>Table 8 "Seismic Combination load Cases according to Equation 2.16, Eurocode"</i>	12
Table 9 "Combination Load Cases, API 650"	13
Table 10 "Design load factors"	14
Table 11 "Undrained Bearing Capacity, Prandtl's solutions, versus undrained cohesion"	23
Table 12 "Drained Bearing capacity in according to the Brinch Hansen Equation for; $c'=0$ kPa, $c'=10$ kPa, $c'=20$ kPa"	24
Table 13 "Laboratory Soil Tests, Basrah site"	27
Table 14 "Oedometer test's data, Basrah site"	28
Table 15 "Minimum Thickness of the Shell courses"	31
Table 16"Summary of the Shell thickness sections"	32
Table 17 "Wind Loads EN 1991 characteristic forces, and ASCE 7-10 design forces (8 meters diameter 10 meters height)"	37
Table 18 "Convective time equations, accordingly to API 650 and EN 1998-4:2006"	39
Table 19 "Impulsive and Convective Mass equations, according to API 650 a EN 1998"	40
Table 20 "Impulsive Mass Height equations for ringwall and slab foundation case in according to API 650 and EN 1998"	41
Table 21 "Convective Mass Height equations for ringwall and slab foundation case in according to API 650 and EN 1998"	42
Table 22 "Overturning Seismic Moment equations, API 650 and EN 1998"	43
Table 23 " Reduction Force coefficients, ringwall and slab foundation, API 650"	45
Table 24"Coefficient of seismic bearing capacity check, Equation 5.42 EN 1998-4:2006"	47
Table 25 " Impulsive and Convective acceleration coefficient, for the case studied, according to API 650 and EN 1998"	48
Table 26 "Case Studied: Overturning Seismic Moment API 650 and EN 1998"	48

Table 27"Combination Factors for Equation 6.1 and Equation 6.2, where ψ and ξ are referred to characteristic loads" EN 1991-1:2003	51
Table 28 "Final Undrained Cohesion due to Surcharge, case studied"	54
Table 29 " Total consolidation rates (UT) in time (t), with vertical drains, case studied"	58
Table 30"Ringwall width section, according to Eurocode Load Combinations, case studied"	59
Table 31 "Ringwall width section according to API 650 Load Combinations, case studied"	60
Table 32 "Combination Load Cases, ACI 318 (PIP STC 01015)"	65
Table 34 "Steel Section Area in according to ACI 318 load combinations, case studied"	70
Table 33 "Steel Section Areas in according to Eurocode load combinations, case studied"	70
Table 35"Undrained bearing capacity, Eurocode, load combinations according to Equation 6.1, case studied"	71
Table 36 " Undrained bearing capacity, Eurocode, load combinations according to Equation 6.2, case studied"	71
Table 37"Undrained seismic stability analysis in according to Eurocode load combinations, case studied"	72
Table 38 "Undrained bearing capacity according to API 650 load combinations, case studied"	72
Table 39 "Drained bearing capacity according to Eurocode load combinations Equation 6.1, case studied"	73
Table 40 "Drained bearing capacity according to Eurocode load combinations Equation 6.2, case studied"	73
Table 41 "Drained bearing capacity according to API 650 load combinations, case studied"	74
Table 42 "Advantages and Disadvantages of Ringwall and Slab concrete foundation (PIP 03020)" ...	B-1
Table 44"Soil Data Borehole 1 BH1 Basra site"	C-1
Table 44 "Soil Data Borehole R9 Basra site"	C-1
Table 45"Soil Data Borehole R10 Basra site"	C-2
Table 46 "Resultant Wind Forces Eurocode (EN 1991-1-4:2004) , case studied"	D-3
Table 47 "Resultant Wind Forces API 650, case studied"	D-4
Table 48"Eurocode Characteristic Load Combinations, case studied"	F-1
Table 49 "Eurocode Characteristic Load Combinations, case studied"	F-2
Table 50 "Eurocode Characteristic Seismic Load Combinations, case studied"	F-3

Table 51 "Eurocode Design Load Combinations, case studied "	F-4
Table 52 "Eurocode Design Load Combinations, case studied "	F-5
Table 53 "Eurocode Design Seismic Load Combinations, case studied "	F-6
Table 54 "API 650Load Combinations, case studied "	G-1
Table 55 "ACI 318 Load Combinations, case studied"	H-1
Table 56 "Combination Loads in according to ACI 318"	H-1

List of Graphs

Graph 1 "Terzaghi's bearing-capacity factors (Craig, 2004)"	21
Graph 2 "Undrained Bearing Capacity in according to the Prandtl's solutions versus undrained cohesion"	23
Graph 3 "Drained bearing capacity in according to: $c' = 0 kPa$, $c' = 10kPa$, $c' = 20kPa$ "	24
Graph 4 "Factors Nq , Nc , $N\gamma$ function of the friction angle ϕ "	24
Graph 5 "Undrained Shear strength from UCS test, Basrah site"	27
Graph 6 "SPT blows data, Basrah site"	28
Graph 7 "Correlation between SPT N values and undrained shear strength c_u , function of plasticity referred to 100 mm diameter specimens (Clayton, Simons, & Matthews, 1982)"	29
Graph 8 "Comparison of convective coefficient time period API 650 and EN 1998"	39
Graph 9 "Impulsive and Convective mass equations, according to Eurocode and API W_i , W_p , W_c are referred to API while m_i m_l m_c are referred to Eurocode"	40
Graph 10 "Centroid of the Impulsive mass, Ringwall case"	41
Graph 11 "Centroid of the Impulsive mass, Slab case"	41
Graph 12 "Centroid of the convective mass, Ringwall case"	42
Graph 13 "Centroid of the Convective mass, Slab case"	42
Graph 14 "Impulsive and Convective Response spectrum, for the case studied, according to API 650 and EN 1998"	48
Graph 15 "Time of the maximum impulsive response, function of the tank's Height Diameter ratio"	49
Graph 16 "Time of the maximum convective response, function of the tank's Height Diameter ratio"	50
Graph 17 "Preload consolidation process"	52
Graph 18 "Relationship between average degree of consolidation and time factor layer drained in both sides (Craig, 2004)"	55
Graph 19 "Horizontal Consolidation rate, for different n ratio (Craig, 2004)"	57
Graph 21 "Effective stresses at the centre of the foundation tank, before and after the excavation, case studied"	62
Graph 21 "Effective stresses at the centre of the foundation tank, before and after the tank's positioning, case studied"	62
Graph 22 "One dimensional consolidation curve showing three independent physical processes- instantaneous strain (initial), primary consolidation and secondary compression (Craig, 2004)"	63

Graph 23 "Wind exposure factor $c_e(z)$ (EN 1991-1-4:2004)"D-1

Graph 24 "Wind coefficient of External Pressure (EN 1991-1-4:2004)"D-3

Definitions

Before starting it is important to give some definitions which will be used in the thesis.

Theory: is a statement which seems to be true by the preponderance of a scientific evidence, but cannot be proven absolutely. In according to the topic treated in this thesis, it may be referred to the reliability theory used by the LSD method, to establish loads and resistances factors, or the bearing capacity theory.

Method: a particular procedure (sequence of formal steps) to solve or resolve a problem, especially a systematic or established one; may be referred to the Limit State Design (LSD) or Allowable Stress Design (ASD) method.

Criteria: a standard on which a judgement is based on, they basically refer to the method used. Generally in a method different criteria have to be full filled. The criteria are referred to structural performance criteria that have to be established before the start of the project. For example adequate strength, stiffness, durability etc. They refers for the LSD method to the Ultimate Limit State (ULS) or Serviceability Limit State (SLS). The main goal for LSD and ASD method is of ensuring a safe structure and ensuring the functionality of the structure. For the API 650, the criteria are based on stability criteria for the foundation, based on the allowable soil bearing capacity. It is similar to the LSD criteria of the Eurocode.

E.g. In decision making the Multicriteria method, uses different criteria in according to the project studied.

Approach: it can be used to establish a criteria. For instance referring to the Eurocode, to assess two ULS criteria i.e. STR & GEO three approaches can be used, they are based on how the partial factors are applied to action or ground strength parameters. For the API 650, the approach is based on a singular factor of safety, by performing the bearing capacity analysis by using unfactored load and ground resistances, and verifying the equilibrium.

Condition: by definition is the state on which something exist; may be refer to a degree of loading or other actions on the structure. Conditions are states for which is possible to apply a particular criterion.

Code: is a model, a set of rules that knowledgeable people recommend for other to follow. It is not a law but can be adopted into a law.

Standard: tends to be a detailed elaboration, set of technical definitions and guidelines, to meet a code

1. Introduction

To develop a project in foreign countries, it is possible to use different standards or codes, which if are properly followed, give an acceptable level of reliability. However the different usage can generate different section dimensions and also reliability of the structure can differ. The thesis has the aim to compare the design of a foundation for an oil tank according to the European code and the American standard by pre-dimensioning the structural section.

1.1 Overview

The thesis starts with a description of the methodologies used by the norms, in particular of the difference between each other in term of the method used. The API 650 uses the “old design method” called Allowable Stress Design (ASD) while the Eurocode uses the Limit State Design (LSD) method. The two methodologies are different in the way that uncertainties are dealt with and how the load combinations are considered. Regarding the LSD (for foundation design), five Ultimate Limit States (ULS) are used which are presented and discussed, particularly on how they can be applied to a tank structure. More attention will be focused on two ULS:

- Failure or excessive deformation of the ground (GEO)
- Internal failure or excessive deformation of the structure (STR).

These two limit states are divided into three approaches which will be presented and discussed.

- Design approach one (DA1), is split into two “sub-approaches”, which have to be both fulfilled. The first, of the two, applies factors to the loads while the second reduces the material strength resistances.
- Design approach two (DA 2), increases the loads and reduces the resistance failure mechanism of the foundation.
- Design approach three (DA 3), applies factors to loads and material resistance, but different load factors are used if structural or geotechnical loads are considered.

The different approaches constituting the Eurocode allows each country of the European community to choose which approach is used, then in an international environment of construction one approach rather than the other can be picked by the engineer. It is advisable to discussing and make an overview of them. The ASD method will also be presented. A general overview with the advantages and disadvantages on using ASD or LSD is presented.

Wind, seismic and dead loads acting on the foundations have been calculated. Regarding the earthquake and wind load, they have been calculated using the methods in the Eurocode and API 650 and comparing the results according to the case studied.

An overview of the bearing capacity failure mechanisms that could occur is presented, focusing in particular on the general shear failure mechanism (Brinch Hansen equation). In fact, to assess the bearing capacity in the Eurocode the general shear failure mechanism is considered according to the Brinch Hansen equation. The Brinch Hansen equation will be presented by using the analytical solution of the upper and lower limit method introduced by Prandtl and Terzaghi going through all the theory until the widely used Brinch Hansen equation (Gudehus, 1981 & Coduto, Yeung, & Kitch, 2011).

The comparison of the norms is presented considering a particular case studied, one of the five foundations typologies which can be used for tank will be pre-dimensioned, (in according to PIP

STE03020): concrete ringwall, crushed stone ring wall, concrete slab, compacted granular fill and pile foundation. The case studied will foresees the predimensioning of a ringwall foundation.

Between the two norms, the comparison has been focused on:

- Load Combination
- Method used
- Criteria
- Approach
- Simplicity of using a norm rather than another
- Section's foundation

1.2 Norm used

Generally norms for the European and American designers are as follows.

Design in accordance to the European norms can be done with the Eurocodes. The Eurocodes used for the load determination are:

- EN 1990:2002 *"Basis of structural design"*,
- EN 1991-4:2004 *"Actions on structures - General actions – Part 1-4: Wind actions"*. With regards to the steel structural part, in according to the soil structure interactions, the reference Eurocode are
- EN 1993 - 4 - 2 *"Design of steel structure – Part 4 – 2: Tanks"*
- EN 1993 - 1 - 6 *"Design of steel structures - part 1 - 6: General - Strength and Stability of Shell Structures"*.

For the geotechnical design shall be used:

- EN 1997 - 1:2004 *"Geotechnical design - Part 1: General Rules"*.

Regarding the seismic analysis it is required to use

- EN 1998-1:2004 *"Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings"*
- EN 1998-4:2006 *"Design of structures for earthquake resistance – Part 4: Silos, tank and pipelines"*.
- EN 1998-5:2004 *"Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects"*

Regarding the American norms, the API norms can be used, in particular:

- API 650:2012 *"Welded Steel Tanks for Oil Storage"*
- API 653:2003 *"Tank Inspection, Repair Alteration, and reconstruction"*.

Moreover for the wind load and the snow load (this last one is not considered for the case studied because the area is not affected by snow precipitations)

- ASCE 7 *"Minimum design Loads for Building and Other Structures"*
- ACI 318 *"Building Code Requirements for Reinforced Concrete"*

A good overview of the American norms is given by guidelines, which can be used as a reference for the design steps of a foundation:

- PIP STE03020:2005 *"Guidelines for Tank Foundation Design"*

- PIP 01015 “*Structural Design Criteria*”.

1.3 Structure of the thesis

The second chapter describes the Allowable Stress Design and the Limit State Design methods used in the API 650 and Eurocode respectively. After that, the five ULS approaches of the Eurocode and the one of the API 650 will be presented and the differences between them are discussed. In particular regarding the Eurocode, the attention is focused on two Ultimate Limit State (GEO & STR).

Third chapter describes three bearing capacity failure mechanisms, with particular focus on the general shear failure mechanism, and the explanation of the Brinch Hansen theory. The factors in the Brinch Hansen are described also in according to the dependency with the strength properties of the soil.

The fourth chapter introduces the case study, in particular this is for a tank of 8 meters in diameter and 10 meters height. In this chapter an overview of the foundations types for a tank will be presented, shell design will be also designed in according to the method presented in API 650.

In the fifth chapter wind and the seismic loads are calculated in accordance to the two norms considered. The results will be compared in terms of loads and methods used to asses them.

In the sixth chapter the calculation of the foundation in according to a particular case studied is presented. Calculations for load combinations are presented, in according to foundation sizing and bearing capacity assessment.

The seventh chapter evaluation of the results of the thesis is presented:

- Overview of the European and American Norm
- Methodologies used
- Wind and Seismic Load comparison
- Load combination comparison
- Eurocode approaches
- Steel Reinforcement sections

Chapter eight foresees recommendations in according to the case studied to perform a better design.

In the ninth chapter conclusions of the comparison between the methods and approaches of the norms are given.

2. Methodologies

During a design of a structure there are uncertainties which the designer has to deal with. The uncertainties can be related to variable material properties e.g. nominal capacities and resistance factors or uncertainties related to loads e.g. design loads and load factors. To give some examples the capacity of the soil are the material properties e.g. friction angle and cohesion; resistance factors are related to failure mechanisms e.g. bearing capacities, sliding resistance etc. Regarding the loads acting on the structure they are generally classified as permanent and variable loads. In the norms evaluated, two main design methodologies have been encountered: the allowable stress design (ASD) and the load resistance factor design method (LRFD). The ASD is used by the API 650, it deals with the uncertainties by using a single factor of safety, while the LRFD, used in the Eurocode, deals with multiple factors. The Eurocode uses the Limit State Design (LSD) method where the factors are applied to loads, resistances soil strength properties and resistance failure mechanism considered. For geotechnical design, the European reference norm is EN 1997-1, this code is not used just for tank design but it covers all types of structures, it is a binding document for all the European countries. For how concern USA, a complete formal geotechnical design code of practice, addressing all aspects of geotechnical engineering, does not exist on national scale (Allen, 2012). However a code developed by the Association of Highway and Transportation Officials (AASHTO), which uses the LRFD methodology can be considered as the equivalent Limit States Design method (LSD) but it deals with transport structures, including a geotechnical section and it is not considered for the design of all structural typologies.

2.1 Load and Resistance Factor Design Method

The Eurocode uses LSD a particular type of the “Load and Resistance Factor Design” method. This design method uses factors that are applied to structural loads (or to effects of the loads), on the resistance mechanisms and on the geotechnical parameters. Generally when a limit state of rupture or deformation is considered it should be verified that the design values of the effect of the action are smaller than the design resistance of the soil. The factors applied are based on the reliability theory, which is the probability of failure.

2.1.1 Mean and Characteristic value

In geotechnical design, the engineer generally deal with several test data of a soil property. The first attempt is by defining a mean value X_m which tell us about the average value of the soil property. If there are n numbers of soil properties x_n the mean value X_m is defined as:

$$X_m = \frac{x_1 + x_2 + \dots + x_n}{n} \quad \text{Equation 2.1}$$

The n values can be approximated by using a probability density function as it is shown in Figure 1. The probability density function $f(x)$ is the “bell function”, whose properties are:

1. $f(x) \geq 0$ for all x

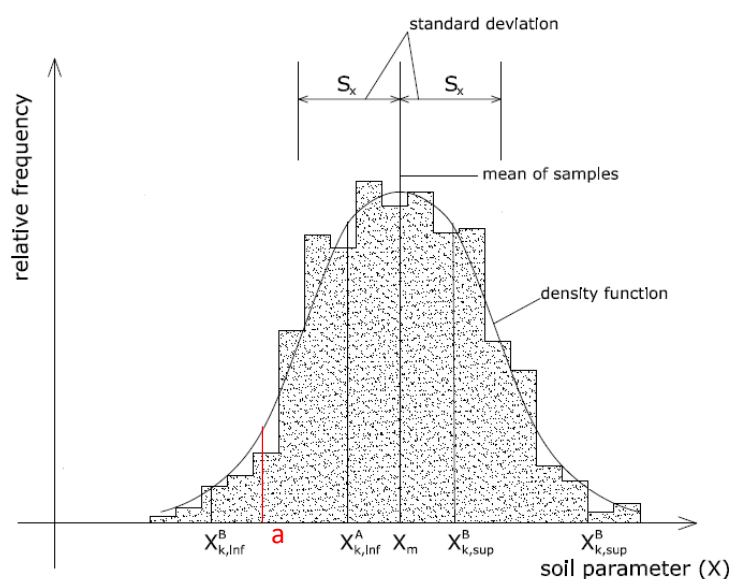


Figure 1 "Probability density function distribution (Bashor, Kareem, & Moran, 2009)"

2. $\int_{-\infty}^{+\infty} f(x) dx = 1$
3. $p(a \leq x) = \int_x^{+\infty} f(x) dx$ *reliability*
4. $p(x \leq a) = \int_{-\infty}^a f(x) dx$ *risk of failure*

Where p is the probability.

In according to the Eurocode: the characteristic value (a particular “a” value of Figure 1) should be derived such that the calculated probability of a worse value is not greater than 5% (EN 1997-1:2004, p.28).

In according to Figure 1, the 95 percentile value of all the soil strength property defined by several test data, is selected as the characteristic value X_k . Using this soil property for design it means that there is 1 in 20 chance that this soil property in the soil mass will be smaller.

$$p(X_k) = \int_{X_k}^{+\infty} f(x) dx \geq 0.95 \quad \text{Equation 2.2}$$

In according to the Eurocode, the calculation of the characteristic value for geotechnical parameters shall account of:

- Geological and other background information
- The extent of the field and laboratory investigations
- Type and number of samples
- The extent of the zone of ground governing the behaviour of the geotechnical structure at limit state being considered
- The ability of the geotechnical structure to transfer loads from weak to strong zones in the ground

Methods to assess the characteristic values of the soil properties are generally incorporated in the national annexes they differ in according to the type of foundation e.g. shallow footing or pile foundations etc.

2.2 Limit State Design method

This paragraph describes the EN 1997-1:2004 design method i.e. LSD (Limit State Design method). The LSD method consists of two limit states, the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). The difference between them is that the Ultimate Limit State (ULS) is associated with failure of the structure or failure due to soil mechanisms. Serviceability Limit State (SLS) corresponds to a condition beyond which specified service requirements for a structure or structural member are no longer met, the structure generally loses its serviceability and this mechanism is not necessary linked to failure mechanisms. The geotechnical analysis is performed by using design values for forces and geotechnical properties derived from the respective characteristic values. In the following paragraphs the determination of these design values are defined.

2.2.1 Design Values of action

The design values of an action are determined from the EN 1990:2002. The design value $F_d = \gamma_F F_{rep}$ with the representative force $F_{rep} = \Psi F_k$. F_k is the characteristic value of the action, the value of Ψ convert the characteristic value to the representative value F_{rep} , it is the combination factor for the variable and permanent loads, it defines the probability of occurrence of the loads. The value of γ_F

(partial factor of an action) should be defined in accordance to the favourable or unfavourable nature of the force, the factors change for permanent and variable loads.

2.2.2 Design Values of geotechnical parameter

The determination of the design value of geotechnical parameter X_d , has to be derived from the characteristic values X_k , (Equation 2.2).

$$X_d = X_k / \gamma_M \quad \text{Equation 2.3}$$

- γ_M is the partial factor, it changes in according to the geotechnical property considered.

2.2.3 Eurocode ULS

In EN 1997-1:2004, five ultimate limit states are encountered, (the check of a limit state rather than another is in according to the project considered).

1. Loss of equilibrium of the structure or the ground considered as a rigid body, in this case the strength of the structural materials and the ground are significant in providing resistance. (EQU) (this method is mainly relevant in structural design and it is applied in rare cases such as rigid foundation bearing on rocks);
2. Internal failure or excessive deformation of the structure or structural elements, including e.g. footing, piles or basement walls, in this case just the strength of the structural elements is providing resistance (STR);
3. Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO);
4. Loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical action (UPL)
5. Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).

2.2.3.1 GEO and STR Limit State

Regarding GEO and STR ULS, the manner of how combinations between the resistances, effects of the forces and soil parameters are considered is function of three approaches. The approaches changing in the way of how they distribute the partial factors between: actions, effects of the actions, material properties and resistances. The use of an approach rather than another is generally specified by national codes of the European countries.

In the following paragraphs the three approaches are explained and the following terminology is used:

- A for actions and effects of actions
- M for soil parameters
- R for the resistances mechanisms

Partial factors used for the three approaches are summarized in Table 10.

It should be guaranteed that the design value of the effect of action E_d is smaller than design value of the resistance mechanism in according to the foundation's failure mechanism R_d :

$$E_d \leq R_d \quad \text{Equation 2.4}$$

In according to (EN 1991:2002, 1.5.3.2) the effect of action is so stated:

Effect of actions (or Action Effect) on structural members, (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation)

For resistance in according to (EN 1990:2002, 1.5.2.15) and (EN 1997-1, 1.5.2.7) resistance is defined as:

Capacity of a component, or cross-section of a component of a structure, to withstand actions without mechanical failure

Basically the effects of a force can be calculated by different way, in the equations presented below Equation 2.5 and Equation 2.6, the Eurocode suggests to apply partial factors to the effects (E) or to the forces (F_{rep}) respectively. For the case of a tank, EN 19971:2004, it is suggested to apply the factors directly to the effects of the action, because it gives more reasonable results. Then:

$$E_d = E \left\{ \gamma_F F_{rep}; \frac{X_k}{\gamma_M}; a_d \right\} \quad \text{Equation 2.5}$$

Or

$$E_d = \gamma_E E \left\{ F_{rep}; \frac{X_k}{\gamma_M}; a_d \right\} \quad \text{Equation 2.6}$$

Where:

- γ_E Partial factor for the effect of the action
- γ_F Partial factor for the an action
- F_{rep} Representative value of an action
- X_k Characteristic value of a material property
- γ_M Partial factor for a soil parameter (material property), also accounting for model uncertainties
- a_d Design value of geometrical data

2.2.3.1.1 Design resistances

Design resistance values can be calculated as following, where partial factors may be applied either to the ground properties (X) or resistance (R) or to both, as follow:

$$R_d = R \left\{ \gamma_F F_{rep}; \frac{X_k}{\gamma_M}; a_d \right\} \quad \text{Equation 2.7}$$

Or

$$R_d = R \left\{ \gamma_F F_{rep}; \frac{X_k}{\gamma_M}; a_d \right\} / \gamma_R \quad \text{Equation 2.8}$$

Or

$$R_d = R \left\{ \gamma_F F_{rep}; X_k; a_d \right\} / \gamma_R \quad \text{Equation 2.9}$$

Where γ_R is the resistance factor for a resistance value.

2.2.3.1.2 Design Approach 1

By using this approach it should be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 1: $A1$ "+" $M1$ "+" $R1$

Combination 2: $A2$ "+" $M2$ "+" $R1$

(The symbol "+" used in the following definitions means combination)

In this approach, partial factors are applied to action and ground strength parameters.

APPROACH 1							
	A1	A2		M1	M2	R1	
Permanent Unfavourable Load	1.35	1.00	c'	1.0	1.25	Bearing ultimate	1.0
Variable Unfavourable load	1.5	1.3	tan ϕ'	1.0	1.25	Table 1 "Design load partial factors, Approach 1"	
Permanent Favourable Load	1.0	1.0	c _u	1.0	1.40		
Variable Favourable Load	0.0	0.0	γ_{γ}	1.0	1.00		
			q _u	1.0	1.00		

2.2.3.1.3 Design Approach 2

It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination:

Combination: A1 "+" M1 "+" R2

(The symbol "+" used in the following definitions means combination)

In this approach, partial factors are applied to actions or to the effects of actions and to ground resistances. The combination factors are shown in Table 10.

Such as for the case in the approach 1, $\gamma_F = 1$ and $\gamma_E \neq 1$.

The effect of the forces and the material parameters used in the combination of approach 2 are the same of the combination 1 of the approach 1 however in this approach the resistance values used to verify the failure mechanism changes (R2 is used instead R1).

APPROACH 2					
	A1		M1		R2
Permanent Unfavourable Load	1.35	c'	1.0	Bearing ultimate	1.4
Variable Unfavourable load	1.5	tan ϕ'	1.0	Table 2 "Design load partial factors, Approach 2"	
Permanent Favourable Load	1.0	c _u	1.0		
Variable Favourable Load	0.0	γ_{γ}	1.0		
		q _u	1.0		

2.2.3.1.4 Design Approach 3

It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination:

Combination: (A1 or A2) "+" M2 "+" R3

(The symbol "+" used in the following definitions means combination)

In this approach, partial factors are applied to actions (or effects of actions) but for this approach different factors are applied in according to the case when the actions (or effects of actions) is from the structure A1 or from ground forces (or effects of forces) A2 e.g. surcharge load etc..

Such as for the case in the approach 1, $\gamma_F = 1$ and $\gamma_E \neq 1$, then Equation 2.6 is used.

However different factors of the resistance failure mechanism are applied compared to the previous approaches.

APPROACH 3						
	A1	A2		M2		R3
	Structural Actions	Geotechnical Actions				
Permanent Unfavourable Load	1.35	1.00	c'	1.25	Bearing ultimate	1.0
Variable Unfavourable load	1.5	1.3	tan ϕ'	1.25	Table 3 "Design load partial factors, Approach 3"	
Permanent Favourable	1.0	1.0	c_u	1.40		
Variable Favourable Load	0.0	0.0	γ_v q_u	1.00 1.00		

2.2.3.2 EQU Limit State

The static equilibrium of the structure it shall be verified that:

$$E_{dst;d} \leq E_{stb;d} + T_d \quad \text{Equation 2.10}$$

Where:

- $E_{dst;d}$ design value of the effect of destabilising actions
- $E_{stb;d}$ design value of the effect of stabilising actions
- T_d Design value of the total shearing resistance that develops around a block of ground in which a group of tension piles is placed, or on the part of the structure in contact with the ground

Basically there are factors applied to the actions and they are divided in according if they are favourable or unfavourable, factors are also applied to the soil parameters when the failure is considered.

In according to EN 1997-1:2004, static equilibrium EQU is mainly relevant in structural design, and it is applied in rare cases, such as rigid foundation bearing on rock.

2.2.3.3 UPL Limit State

The uplift limit state is carried out to check if the sum of the permanent vertical action of a structure $G_{stb;b}$ plus any additional resistance to uplift R_d is smaller or equal to the combination of destabilizing permanent and variable vertical action $V_{dst;d}$, due to uplift forces.

$$V_{dst;d} \leq G_{stb;b} + R_d \quad \text{Equation 2.11}$$

2.2.3.4 HYD Limit State

This is the limit state of failure due to heave seepage of water in the ground. In according to this limit state, the destabilizing total pore water pressure $u_{std;d}$ at the bottom of the column, should be less than or equal to the stabilizing total vertical stress $\sigma_{stb;d}$:

$$u_{std;d} \leq \sigma_{stb;d} \quad \text{Equation 2.12}$$

2.2.4 Eurocode SLS

In case of a serviceability state verification, it shall require that the effect of the actions E_d should be smaller than the limiting design values of the relevant serviceability criterion C_d .

$$E_d \leq C_d \quad \text{Equation 2.13}$$

In according to EN 1997-1:2004, values for the partial factors for serviceability limit states should normally be taken equal 1.0. Then the analysis is performed by using characteristic values of the soil properties and the forces, or effect of the forces.

2.2.5 Load Combination for ULS

The general idea in the Eurocode for the load combination, is that the design value of the effects of the action E_d shall be determined by combining the effect of the actions acting in according to a combination scheme. With reference to EN 1990, combination loads are done to take into the probability of occurrence, in particular it is unlikely that earthquake and wind load act simultaneously so they are considered separately. In according to all the limit states, EN 1990:2002, gives a general formulation of the combination of the loads. However in according to the STR and GEO limit state less favourable combination should be used.

2.2.5.1 Tank and Silos Load Combinations

Combination load is performed in according to EN 1991-4:2003, which is the Eurocode for actions, for Tanks and Silos.

In according to the limit states STR and GEO the Eurocode require to use Equation 2.14 and Equation 2.15 taking the less favourable combination load.

The combination of the loads should be done according to the following equation:

$$\left\{ \begin{array}{l} \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \\ \sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \end{array} \right. \quad \begin{array}{l} \text{Equation 2.14} \\ \text{Equation 2.15} \end{array}$$

Where:

- $G_{k,j}$ are the permanent loads
 - Tank's weight
 - Roof
 - Foundation weight
 - Soil inside the ring
 - Stabilizing soil
- $Q_{k,1}$ Variable loads
 - Liquid tank
 - Wind
 - Seismic

Factors for combination of variable loads are:

- ψ_0 factor for combination of a variable load
- ψ_1 factor for frequent value of a variable load
- ψ_2 factor for quasi-permanent value of a variable load

The values of γ and ψ for actions should be taken from EN 1991-4 and from EN 1990:2002. These values can differs in according to the national annexes of each European country, in this thesis, the values provided by the Eurocode will be used.

In load combinations, the coefficient combination factor ψ to account the probability of simultaneous occurrence are taken from Table 4, which has been taken from EN 1991-4:2003. This table is a supplementary clause of EN 1990 for silos and tanks. The EN 1991-4:2003 so states:

“In principle the general format given in EN 1990 for design procedures is applicable. However silos and tanks are different to many other structures because they may be subjected to the full loads from particles solids or liquid for most of their life.”

In other words, EN 1991-4:2003 states that for combination load it is advisable to use the factor for load combination in according to Table 4, rather than the values given in EN 1990, which are referred to all type of structures.

Table 4 "Combinations load Factors, Ultimate Limit State (ULS) EN 1991-4:2003 table a.1"

Short Title	Design Situation/leading variable action	Permanent action		Accompanying variable action 1 (main)		Accompanying variable action 2		Accompanying variable action 3,4 etc	
		Description	ξ_1	Description	$\psi_{0.1}$	Description	$\psi_{0.2}$	Description	$\psi_{0.3}$ $\psi_{0.4}$ etc.
D	Liquid Discharge	Self Weight	0.9	Liquid filling	1.0	Foundation settlement	0.7	Snow, wind, thermal	0.6
								Imposed loads, imposed deformation	0.7
I	Imposed load or deformation	Self Weight	0.9	Liquid Filling	1.0	Imposed Deformation	0.7	Snow, wind, thermal	0.6
								Imposed loads	0.7
S	Snow	Self Weight	0.9	Liquid Filling	1.0	Snow	0.6	Imposed loads	0.7
WF	Wind and full	Self Weight	0.9	Liquid Filling, full tank	1.0	Wind	0.6	Imposed loads	0.7
WE	Wind and empty	Self Weight	0.9	Liquid, empty tank	0.0	Wind	0.6	Imposed loads	0.7

Regarding seismic load combinations:

$$\sum_{j \geq 1} G_{k,j} + A_{ED} + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \quad \text{Equation 2.16}$$

Where:

- A_{ED} is the design value of seismic action $A_{ED} = \gamma_I A_{EK}$
 - o A_{EK} Characteristic value of seismic action
 - o γ_I importance factor

Table 5 "Factors for seismic load combinations (ULS) EN 1991-4:2003"

Seismic Ultimate limit State ULS											
Short Title	Design Situation/leading variable action	Permanent action		Leading Variable Action		Accompanying variable action 1 (main)		Accompanying variable action 2		Accompanying variable action 3,4 etc	
		Description		(See next column "main")		Description	$\psi_{2.1}$	Description	$\psi_{2.2}$	Description	$\psi_{2.3}$ $\psi_{2.4}$ etc.
SF	Seismic action and full silo	Self Weight		Seismic action (earthquake)		Liquid filling, full silo	0.8	Imposed Deformation	0.3	Imposed Loads	0.3
SE	Seismic action and empty silo	Self Weight		Seismic action (earthquake)		Liquid, Empty Silos	0.8	Imposed Deformation	0.3	Imposed Loads	0.3

Table 6 "Combination Load Cases according to Equation 2.14, Eurocode"

<i>D</i>	<i>Liquid Discharge</i>	$\gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} 1.0 F + \gamma_{Q,i} 0.6 (S + W)$
<i>I</i>	<i>Imposed load or deformations</i>	$\gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} 1.0 F + \gamma_{Q,i} 0.6 (S + W)$
<i>S</i>	<i>Snow</i>	$\gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} 1.0 F + 0.6 S$
<i>WF</i>	<i>Wind and Full Tank</i>	$\gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} 1.0 F_h + 0.6 W$
<i>WE</i>	<i>Wind and Empty Tank</i>	$\gamma_{G,j}(D_f + D_L) + 0.6 W$

Table 7 "Combination Load Cases according to Equation 2.15, Eurocode"

<i>D</i>	<i>Liquid Discharge</i>	$0.9 \gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} F + \gamma_{Q,i} 0.6 (S + W)$
<i>I</i>	<i>Imposed load or deformations</i>	$0.9 \gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} F + \gamma_{Q,i} 0.6 (S + W)$
<i>S</i>	<i>Snow</i>	$0.9 \gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} F + 0.6 S$
<i>WF</i>	<i>Wind and Full Tank</i>	$0.9 \gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} F_h + 0.6 W$
<i>WE</i>	<i>Wind and Empty Tank</i>	$0.9 \gamma_{G,j}(D_f + D_L) + \gamma_{Q,1} D_L + 0.6 W$

Table 8 "Seismic Combination load Cases according to Equation 2.16, Eurocode"

<i>SF</i>	<i>Seismic and Full Tank</i>	$(D_f + D_L) + E + 0.8 F_h$
<i>SE</i>	<i>Seismic and Empty Tank</i>	$(D_f + D_L) + E$

Where:

- D_f Dead load of the internal floating roof
- D_L Dead Load: weight of the tank
- E Seismic load
- F Stored Liquid load at design height
- F_h Stored Liquid load for all the height of the tank
- F_p Pressure combination factor, defined as the ratio of normal operating pressure to design pressure, with a minimum value of 0.4.
- H_t Hydrostatic test load
- L_r Minimum roof live load
- P_i Design Internal Pressure
- P_e Design external pressure
- P_t Test pressure
- S Snow load
- S_b Balanced design snow load
- S_u Unbalanced design snow load
- W Wind pressure load

Load factors $\gamma_{Q,n}$ for transient, favourable and unfavourable loads are in according to the Limit state considered Table 10.

2.3 Allowable Design Method

The allowable stress design method is one of the oldest method that has been used by the engineer designers, it foresees to use unfactored loads and unfactored geotechnical design. The unfactored load can be deduced by the definition in the beginning of the API 650, where it is stated that this norm is based on the allowable stress design method. Subsequently from ASCE 7, which is the reference norm for the load determination (ASCE 7-10, p. 239).

“In case of allowable stress design, design specifications define allowable stresses that may not be exceeded by load effects due to unfactored loads...”

Failure mechanism is checked by considering a single safety factor, called Factor of Safety (*FS*). It is easy to use because fewer analyses are generally performed. However this factor is not reliability based because there are not informations about probability of failure.

The general form of the model is:

$$Load \leq \frac{Capacity}{FS} \quad \text{Equation 2.17}$$

The global Factor of Safety *FS* is typically based on experience. This factor generally does not vary from one design to another and then this method tends to uniform all type of projects.

2.3.1 Factor of safety for Bearing Capacity

API 650 prescribes factors of safety that have to be fulfilled against foundation’s bearing capacity instability, they are summarized below:

1. From 2.0 to 3.0 against ultimate bearing failure for normal operating conditions;
2. From 1.5 to 2.25 against ultimate bearing failure during hydrostatic testing;
3. From 1.5 to 2.25 against ultimate bearing failure for operating conditions plus the maximum effect of wind or seismic load.

2.3.2 Load Combinations

Level for uncertainties are generally related to the load combination and also to the determination of them (dead and the transient loads). Specification of the load type are in API 650.

Table 9 "Combination Load Cases, API 650"

API 650 (Load Combination)		
1	Fluid and Internal Pressure	$D_L + F + P_i$
2	Hydrostatic Test	$D_L + (H_t + P_t)$
3	Wind and Internal Pressure	$D_L + W + F_p(P_i)$
4	Wind and External Pressure	$D_L + W + 0.4 P_e$
5	Gravity Loads	$D_L + (L_r \text{ or } S_u \text{ or } S_b) + 0.4 P_e$
		$D_L + P_e + 0.4(L_r \text{ or } S_u \text{ or } S_b)$
6	Seismic	$D_L + F + E + 0.1S_b + F_p(P_i)$
7	Gravity Loads for Fixed Roofs with Suspended Floating Roof	$D_L + D_f + (L_r \text{ or } S) + P_e + 0.4\{L_{f1} \text{ or } L_{f2}\}$
		$D_L + D_f + \{L_{f1} \text{ or } L_{f2}\} + 0.4\{(S \text{ or } L_r) + P_e\}$

Where:

- D_f Dead load of the internal floating roof
 - L_{f1} internal floating roof uniform live load (0.6Kpa)
 - L_{f2} internal floating roof point load, at least two men walking anywhere on the roof applied load of 2.2KN
- D_L Dead Load: weight of the tank
- E Seismic load
- F Stored Liquid load

- F_p pressure combination factor, defined as the ratio of normal operating pressure to design pressure, with a minimum value of 0.4.
- H_t Hydrostatic test load
- L_r Minimum roof live load
- P_i Design Internal Pressure
- P_e Design external pressure
- P_t test pressure
- S Snow load
- S_b balanced design snow load
- S_u unbalanced design snow load
- W Wind pressure load

2.4 Differences

The main topic encounter in the LSD method and not in ASD are:

1. Separation of geotechnical procedures into clearly defined limit states
2. Define what is a load and what is a resistance, by respective safety factors and not by using an overall factor of safety

Moreover a problem related to the safety factor is that a single safety factor is applied to all designs and it is certainly known that uncertainties varies from site to site and thus project to project.

Another difference between the approaches is that in the LSD method there are several factors to account for the load factors, while the uncertainties for the ASD norms are encounter by a single factor.

Since the Eurocode is based on reliability (probability of failure) it is more meaningful than factor of safety which does not tell anything about probability of failure.

Table 10 "Design load factors"

	EUROCODE EN 1997-1:2004					API 650
	EC7 DA 1		EC7 DA 2	EC7 DA 3		
	EC7 DA 1/1	EC7 DA 1/2		structural actions	geotechnical actions	
Permanent Unfavourable Load	1,35	1,0	1,35	1,35	1,0	1
Variable Unfavourable load	1,5	1,3	1,5	1,5	1,3	1
Permanent Favourable Load	1,0	1,0	1,0	1,0	1,0	1
Variable Favourable Load	0,0	0,0	0,0	0,0	0,0	1
c'	1,0	1,25	1,0	1,25		1
$\tan \phi'$	1,0	1,25	1,0	1,25		1
c_u	1,0	1,4	1,0	1,4		1
γ_u	1,0	1,0	1,0	1,0		1
Bearing ultimate	1,0	1,0	1,4	1,0		2 to 3
hydrostatic test						1.5 to 2.25
ultimate+wind+seismic						1.5 to 2.25
Sliding	1,0	1,0	1,1	1,0		load factors in seismic analysis

2.5 GEO and STR approaches considerations

Eurocode allows each country of the European community to use one of the three geotechnical “Design Approaches”. Each nation then published a National Annex (Standard) with the relative approach that has to be used. However for the Eurocode one of the three “Design Approaches” can be chosen. Generally countries which adopt DA2 requires to use also DA3, this is because when a finite element analysis is perform by using DA2, it is difficult to apply factors to quantities which are internal to the numerical analysis, such as active and passive forces and bearing resistance (for spread footing). The use of DA2 and DA3, is similar to the use of DA1, however whereas use of DA2 and DA3 imply to check just one of the two DA1 requires to fulfil both the calculations (Simpson, 2013).

2.6 Summary

This chapter has the aim to emphasize the differences of the methods used by the two norms. In particular it is shown the different theoretical approach, the Eurocode uses the reliability theory to perform the analysis while API 650 method is based on single factor of safety based on experience. It has been also discussed the methodologies encountered during the comparison, in particular it has been presented the LSD and the ASD method. Moreover this chapter has the aim to show the combination loads that have to be considered when we want to design a tank foundation. At the end of this chapter the differences of the two methods (LSD and ASD) have been shown.

This chapter is done to show from a theoretical point of view the differences between the methodologies of the two norms, which will be compared once the bearing capacity of the foundation will be calculated in according to the bearing general shear failure mechanism of the case studied.

3. Shallow Foundation: Ultimate Bearing Capacity

This chapter describes the main mechanisms of failure for shallow foundations. Soil mechanics theory to assess bearing capacity has old basis and the theory started in the earlier of XX century. Brinch Hansen equation for the drained capacity and Prandtl solution for undrained bearing capacity, will be used to compare the bearing capacity calculated by using the two norms, these two equations have been taken as reference because suggested by the EN 1997-1:2004. This chapter has the aim to describe the theory behind the Brinch-Hansen equation from the beginning of its theory until the final formulation, this is done to explain the terms inside the equation. It should be underlined that this equation has a strong dependency with the friction angle, at the end of the chapter a tables and graphs describing dependency of the bearing capacity with the changing of the friction angle and undrained cohesion are presented.

3.1 Failure Mechanisms

Shallow foundations should be verified against two main characteristics:

1. They have to be safe against overall shear failure in the soil that support them
2. They cannot undergo excessive displacements

This chapter is a review of the failure mechanisms related to bearing capacities of shallow foundations.

Considering the overall shear failure in the soil, three types of failure mechanisms can be encountered. The three failure mechanisms are shown in Figure 2, (Braja, 2009).

General shear failure, where the failure surface is well defined and it starts from the edge of the foundation and reaches the ground surface Figure 2 a.

Local shear failure mechanism, where the failure surfaces are well defined just near the foundation and they are missed in the soil mass. In this failure mechanism, shown in Figure 2 b: after relevant increasing of bearing pressure failure surfaces appear on the ground surface.

Punching shear failure, for which at the increasing of the load applied there is a vertical settlement of the foundation with the generation of vertical shear plane along the perimeter of the foundation Figure 2 c.

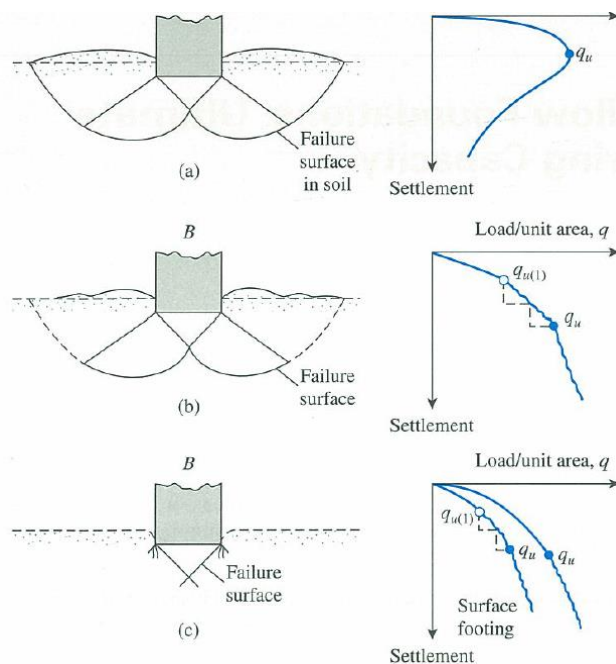


Figure 2 "Modes of bearing capacity failure: (a) general shear failure; (b) local shear failure; (c) punching shear failure" (Braja, 2009)

The general shear failure mechanism is described by the Brinch Hansen equation which makes use of three theories:

1. Method of the plastic equilibrium (Upper and Lower limit)
2. Method of the characteristic lines (Prandtl)

3. Method of global equilibrium (Coulomb)

These three methods will be discussed in this chapter in accordingly to their application (Gudehus, 1981).

3.2 Undrained Case

The method of plastic equilibrium uses the theorem of the upper a lower limit. This method considers the equilibrium of two block; relatively to Figure 3. The explanation of this method has been done in according to (Craig, 2004) and (Gudehus, 1981).

The plastic equilibrium method is applied to a shallow foundation of length B, subjected to a vertical load Q, acting on a homogenous soil with cohesion c_u and with a lateral surcharge p_0 acting on the sides of the foundation. Applying the upper limit it is possible to assume that the failure of the soil mass involves two blocks I and II, which move respect to each other along a surface S, Figure 3.

If the vertical load Q produces a vertical settlement δ , block I moves along the surface S_I of $\delta/\text{sen } \alpha$, similarly block II moves along of a quantity $\delta/\text{sen } \alpha$ along surface S_{II} , then the two block move one respect to the other of 2δ .

The internal work of the system can be expressed as:

$$\frac{c_u B \delta}{\cos \alpha \sin \alpha} + c_u B 2\delta \text{tg} \alpha + \frac{c_u B \delta}{\cos \alpha \sin \alpha} = \frac{2B \delta c_u}{\cos \alpha} \left[\frac{1}{\sin \alpha} + \sin \alpha \right] \quad \text{Equation 3.1}$$

The external work is given by:

$$Q\delta - p_0 B \delta \quad \text{Equation 3.2}$$

Putting equal the internal and external works and solving respect Q:

$$\frac{Q}{B} = q = \frac{2c_u}{\cos \alpha} \left[\frac{1}{\sin \alpha} + \sin \alpha \right] + p_0$$

$$\text{Equation 3.3}$$

The minimum value of q is for $\alpha = 35.3$ and $q_f = 5.66c_u + p_0$. By increasing the number of blocks there is a convergence of the values, in according to Figure 3, for six blocks $q_f = 5.18c_u + p_0$.

The lower limit theorem can be applied when the static equilibrium of the system is studied. The mechanism is linked to the concept that a set of external forces is in equilibrium with an internal system. In

according to the soil mass can be divided into two zones I and II, on the zone I acts a vertical surcharge p_0 and on the zone II acts an uniform bearing pressure q . It should be guaranteed that between the two zones there is an equilibrium and that in each zone the shear stresses are lower than c_u . By using

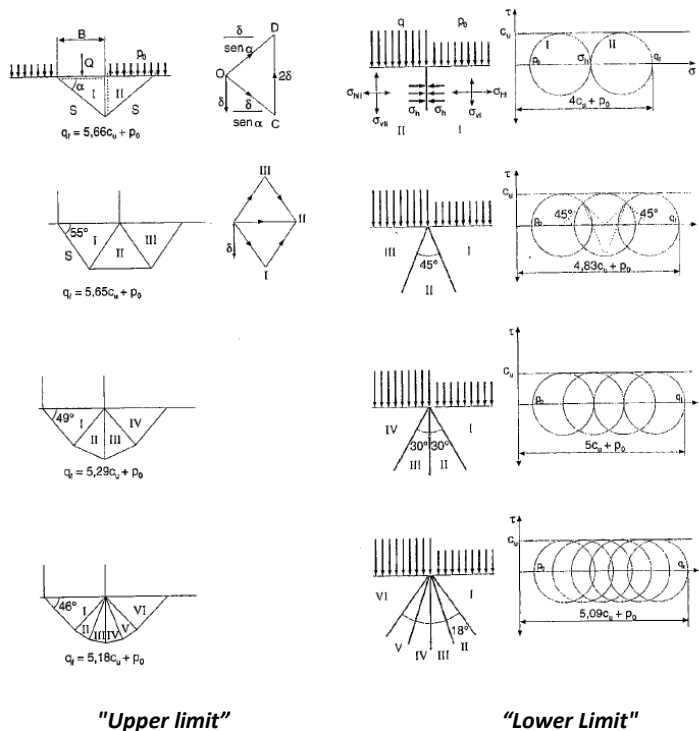


Figure 3 "Upper and Lower limit solutions Undrained Case" (Craig, 2004)

the Mohr diagrams, a vertical pressure p_0 is acting on zone I which can be defined as principal pressure on the Mohr diagram. For the hypothesis of rupture in the zone I, there will be an horizontal pressure σ_{hI} bigger than p_0 and then it is possible to draw the Mohr's circle with a radius equal to c_u . Since the block I and II have to be in equilibrium, along the contact surface $\sigma_{hI} = \sigma_{hII}$ and along this contact plane there are not shear forces, than they are principal stresses. Doing still the hypothesis that the failure occurs in the zone II, the vertical stress $\sigma_{vII} > \sigma_{hII}$; it is possible to draw the Mohr circle with a radius c_u . For equilibrium the $q_f = 4c_u + p_0$, this is the lower limit of rupture.

In according to Figure 3, soil block can be increased in number, increasing their number until six $q_f = 5.09 c_u + p_0$. It has been seen that the exact solution for increasing number of block is $q_f = 5.18c_u + p_0$. The exact solution has been founded by Prandtl considering a soil mass weightless, and considering the only resistance due to the undrained cohesion, $q_f = 5.14 c_u + p_0 = (2 + \pi)c_u + p_0$, defining $(2 + \pi) = N_c$.

$$q_f = N_c c_u + p_0 \quad \text{Equation 3.4}$$

The Eurocode uses a solution modified for a more general case, when the load has an inclination and/or the foundation is inclined.

$$\frac{R}{A'} = (\pi + 2)c_u b_c s_c i_c + q \quad \text{Equation 3.5}$$

Where:

- A' design foundation effective area = $B'L'$
- B foundation width
- B' effective foundation width = $B - 2e$
- b_c is the inclination factor of the foundation $b_c = 1 - \frac{2\alpha}{\pi+2}$
- i_c the inclination factor of the load, caused by an horizontal load H;
 - $i_c = \frac{1}{2} \left(1 + \sqrt{1 - \frac{H}{A'c_u}} \right)$
- L foundation length
- L' effective foundation length = $L - 2e$
- q overburden or surcharge pressure at the level of the foundation base
- s_c is the shape factor of the foundation
 - $s_c = 1 + 0.2 \left(\frac{B'}{L'} \right)$ for rectangular shape
 - $s_c = 1.2$ for circular shape
- R vertical load
- α inclination of the foundation with the horizontal

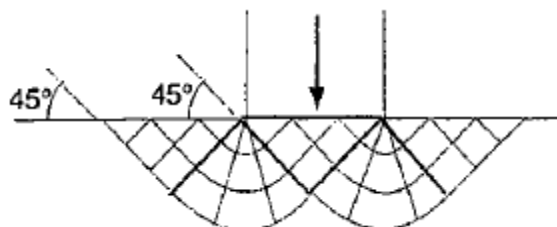


Figure 4 "Prandtl mechanism of failure (Craig, 2004)"

With $H \leq A'c_u$

3.3 Drained Case

In condition of complete drainage, the shear resistance is given by the effective parameters c' and ϕ' and the effective pressures q' and p_0' . Drained analysis method has been described in according to (Craig, 2004). The equilibrium problem can be solved by applying the upper and the lower limit method. Using the lower limit, it is possible to consider a granular material with $c' = 0$ and with friction angle ϕ' . By dividing the system in two blocks zone I and zone II, on top of which is acting the surcharge p_0' and the vertical uniform pressure q respectively. In according to Figure 5, considering the Mohr diagram with the rupture lines $\tau = \sigma' \tan \phi'$ there is a vertical pressure acting on the zone I which can be considered the lower principal pressure. By drawing the Mohr circle, passing for p_0' and tangent to the Coulomb failure line, the corresponding horizontal pressure between the contact surface of block I and II is:

$$\sigma'_{hl} = p_0' \frac{1 + \sin \phi'}{1 - \sin \phi'} = p_0' t g^2 \left(45 + \frac{\phi'}{2} \right)$$

Equation 3.6

Which for equilibrium $\sigma'_{hl} = \sigma'_{hII}$, then it is possible to draw the Mohr circle of the block II, by considering it tangent to the Coulomb failure line, and finding the vertical pressure of the zone II, $\sigma'_{vII} = q'_f$.

$$q'_f = \sigma'_{hl} \frac{1 + \sin \phi'}{1 - \sin \phi'} = p_0' \frac{1 + \sin \phi'}{1 - \sin \phi'} = p_0' t g^2 \left(45 + \frac{\phi'}{2} \right) = p_0' N_q$$

Equation 3.7

The analysis can be done considering more blocks, and then more discontinuity surfaces, the orientation of the discontinuities can be optimized and it depends from the friction angle ϕ .

It is possible to see that increasing the number of blocks, N_q increases and q'_f as well. Repeating the analysis for a cohesive soil:

$$q'_f = p_0' \left(\frac{1 + \sin \phi'}{1 - \sin \phi'} \right)^2 + c' \cot \phi' \left[\left(\frac{1 + \sin \phi'}{1 - \sin \phi'} \right)^2 - 1 \right] = p_0' N_q + c' N_c$$

Equation 3.8

N_c is linked to N_q by the relationship $N_c = (N_q - 1) \cot \phi'$.

There are not exact solutions which give the exact bearing capacity of a shallow foundation with surcharge p_0 and considering a soil mass with own weight γ .

A practical solution is the one given by Terzaghi, the solution is for the case of a shallow foundation with a centred load. This solution uses the limit equilibrium theory and it consists to find by attempts the failure surface more critical, Figure 6. For critical is meant, between all the failure surfaces, the one that can be developed during a failure mechanism, the one which gives the minimum value of bearing resistance. The method foresees to apply the global equilibrium theory to the soil mass restrain in the failure surface.

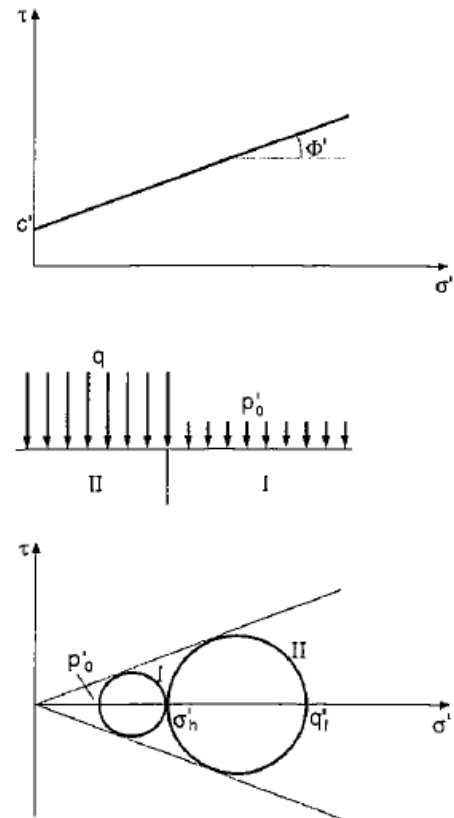


Figure 5 "Lower Limit solution Drained Case" (Craig, 2004)

If the base of the foundation is rough, the friction and the bound between the soil and foundation will not allow to the soil to expand laterally, then the soil in the wedge ABD will remain in an elastic state and it penetrate inside the soil mass. Terzaghi supposed that the two surfaces of the wedge form an angle ϕ with the horizontal plane and the resistance is given by the weight of the soil γ , the surcharge q and the cohesion c . The contribution of each of these parameters can be calculated separately and then summed up together.

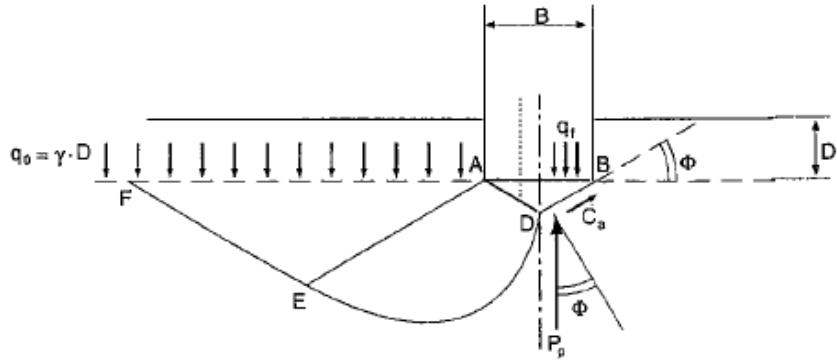


Figure 6 "Terzaghi mechanism of failure" (Craig, 2004)

He supposed that the wedge ADB, cannot penetrate into the soil mass until the pressure on its sides is equal to the passive pressure and then he calculated the critical pressure by using a static equilibrium. At failure the pressure on each surface, AD and BD is equal to the resulting force due to the passive pressure and the force due to cohesion. The resultant of the force acts forming an angle ϕ with the normal on the sliding surface, then it acts vertically.

Zone ADE is the Prandtl's radial zone and AEF is the Rankine passive zone, this slip lines in this zone make an angle of $\pm(45 - \phi/2)$ with the horizontal.

If the weight of the soil is neglected, the resultant force is:

$$Q_f = 2P_p + 2C_a \sin \phi = 2P_p + BC \, tg\phi \quad \text{Equation 3.9}$$

The determination of the passive force P_p is determined by using the method of the global limit equilibrium given by Coulomb. The force P_p can be divided in two components $P'_p + P''_p$. P'_p represents the resistance due to the weight of the mass ADEF and the application point of P'_p is situated at one third from the bottom of AD. The component P''_p can be divided subsequently in other two components: P_c due to the cohesion along the failure surface FED and a second due to the surcharge P_q due to the surcharge $q_0 = \gamma D$. Since these last resultants are uniformly distributed, in according to (Craig, 2004) the point of application is located in the middle of the surface AD. Then in the previous equation is possible to use for $P_p = P'_p + P_c + P_q$, having the general relationship of the bearing capacity per unit of length considering a frictional, cohesive soil mass with weight γ .

$$Q_f = B \left(cN_c + q_0N_q + \frac{1}{2}\gamma BN_\gamma \right) \quad \text{Equation 3.10}$$

Where:

$$N_c = \frac{2P_c}{Bc} + tg\phi$$

$$\text{Equation 3.11}$$

$$N_q = \frac{2P_p}{\gamma DB}$$

$$\text{Equation 3.12}$$

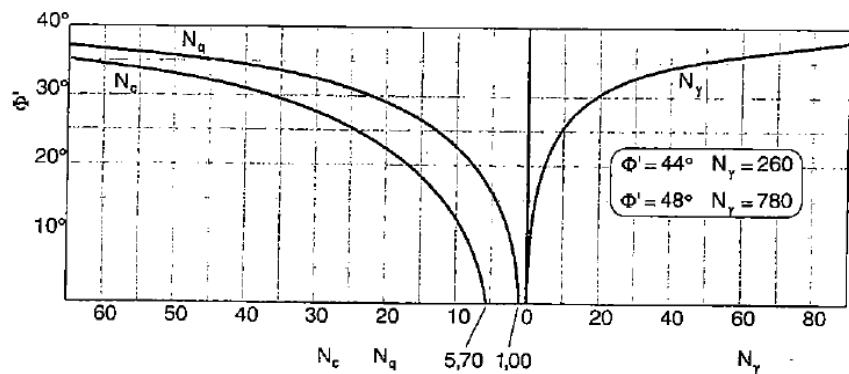
$$N_\gamma = \frac{4P'_p}{\gamma B^2}$$

$$\text{Equation 3.13}$$

The general form for unit of width is:

$$q_f = cN_c + q_0N_q + \frac{1}{2}\gamma BN_\gamma \quad \text{Equation 3.14}$$

The term cN_c represents the contribution of cohesion along the failure surface; q_0N_q represents the stabilizing effect of the surcharge soil at the side of the foundation; the term $\frac{1}{2}\gamma BN_\gamma$ is the contribution of the resistance friction due to the weight of the soil inside



Graph 1 "Terzaghi's bearing-capacity factors (Craig, 2004)"

the sliding surface ADEF. Each term is calculated separately considering: for cN_c that the soil has cohesion and friction angle but not weight, for q_0N_q the material has friction and it is subjected to the surcharge $q_0 = \gamma D$ and for $\frac{1}{2}\gamma BN_\gamma$ the material has weight and friction but not cohesion. The terms N_c , N_q and N_γ are reported in Graph 1.

In according to Graph 1, for $\phi = 0$, the values of $N_\gamma = 0$, $N_q = 1$ and $N_c = 5.70$. Then the bearing capacity of the soil becomes $q_f = 5.70 c_u + \gamma D$.

The Terzaghi's solution, is not exact, basically because the three terms have been calculated separately, with different sliding surfaces and then the effects have been summed together. However the sum of the effects gives a more safe solution.

3.3.1 Brinch Hansen equation

For more general cases, when loads are not centred and they are inclined, Brinch Hansen solution can be used, this solution concerns mostly all the situations. This is the equation used in the Eurocode 7.

3.3.1.1 Drained Case

The design bearing resistance may be calculated from:

$$\frac{R}{A'} = c'N_c b_c s_c i_c + q'N_q b_q s_q i_q + 0.5\gamma'N_\gamma b_\gamma s_\gamma \quad \text{Equation 3.15}$$

In this paragraph the following terms will be used:

- A' design foundation effective area = $B'L'$
- b the design value of the factors for the inclination of the base
- B foundation width
- B' effective foundation width = $B - 2e$
- c' effective cohesion
- D embedment depth
- e eccentricity of the resultant action
- i the inclination factors of the load
- L foundation length
- L' effective foundation length = $L - 2e$
- m exponent in formulas for the inclination factor i
- N the bearing capacity factors

- q overburden or surcharge pressure at the level of the foundation base
- q' the design effective overburden pressure at the level of the foundation base
- s shape factor of the foundation base
- R vertical load
- α inclination of the foundation base to the horizontal
- γ' the design effective weight density of the soil below the foundation level θ direction angle of H
- ϕ' effective friction angle

The effective area A' is referred to the fact that when a load has a certain eccentricity e , in this case it has to be reduced.

The bearing resistance factors:

- $N_q = e^{\pi \tan \phi'} \tan^2 \left(45 + \frac{\phi'}{2} \right)$
- $N_c = (N_q - 1) \cot \phi'$
- $N_\gamma = 2(N_q - 1) \tan \phi'$ where $\delta \geq \phi'/2$

The inclination factors of the foundation base:

- $b_c = b_q - (1 - b_q)/(N_c \tan \phi')$
- $b_\gamma = b_\gamma = (1 - \alpha \tan \phi')^2$

The shape factors of the foundation

- $s_q = 1 + \left(\frac{B'}{L'} \right) \sin \phi'$ for a rectangular shape
- $s_q = 1 + \sin \phi'$ for a square or circular shape
- $s_\gamma = 1 - 0.3 \left(\frac{B'}{L'} \right)$ for a rectangular shape
- $s_\gamma = 0.7$ for a square or circular shape
- $s_c = (s_q N_q - 1)/(N_q - 1)$ for rectangular, square and circular shape

The inclination factors of the load, caused by a horizontal load H;

- $i_c = i_q - (1 - i_q)/(N_c \tan \phi')$
- $i_q = [1 - H/(V + A'c' \cot \phi')]^m$
- $i_\gamma = [1 - H/(V + A'c' \cot \phi')]^{m+1}$

$$m = m_b = \left[2 + \left(\frac{B'}{L'} \right) \right] / \left[1 + \left(\frac{B'}{L'} \right) \right] \text{ when H acts in the direction of B' ;}$$

$$m = m_b = \left[2 + \left(\frac{L'}{B'} \right) \right] / \left[1 + \left(\frac{L'}{B'} \right) \right] \text{ when H acts in the direction of L' ;}$$

In case where the horizontal load component acts in a direction forming an angle θ with the direction of L' ; m may be calculated by:

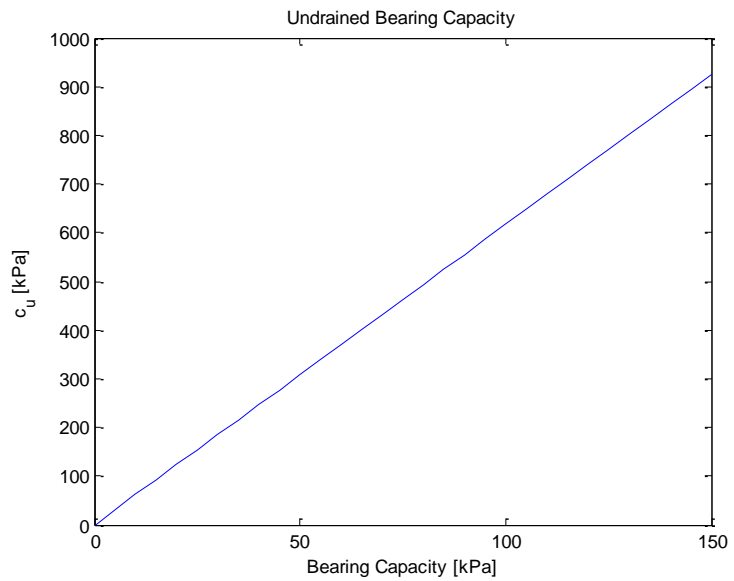
$$m = m_\theta = m_L \cos^2 \theta + m_B \sin^2 \theta$$

3.4 Undrained Case Analysis

The undrained analysis is performed by considering solutions of Equation 3.5, it has been considered a case with not surcharge in the side, the load is vertical, centred and the foundation has a circle shape.

Table 11 "Undrained Bearing Capacity, Prandtl's solutions, versus undrained cohesion"

c _u [kPa]	Bearing capacity [kPa]
0	0
5	30.8
10	61.7
15	92.5
20	123.4
25	154.2
30	185.1
35	215.9
40	246.8
45	277.6
50	308.5
55	339.3
60	370.2
65	401.0
70	431.9
75	462.7
80	493.6
85	524.4
90	555.3
95	586.1
100	617.0
105	647.8
110	678.7
115	709.5
120	740.4
125	771.2
130	802.1
135	832.9
140	863.8
145	894.6
150	925.5



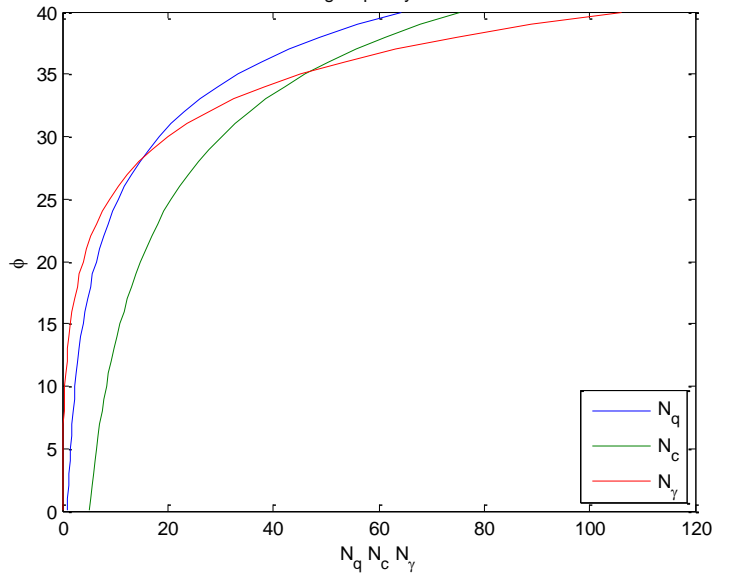
Graph 2 "Undrained Bearing Capacity in according to the Prandtl's solutions versus undrained cohesion"

3.5 Drained Case Analysis

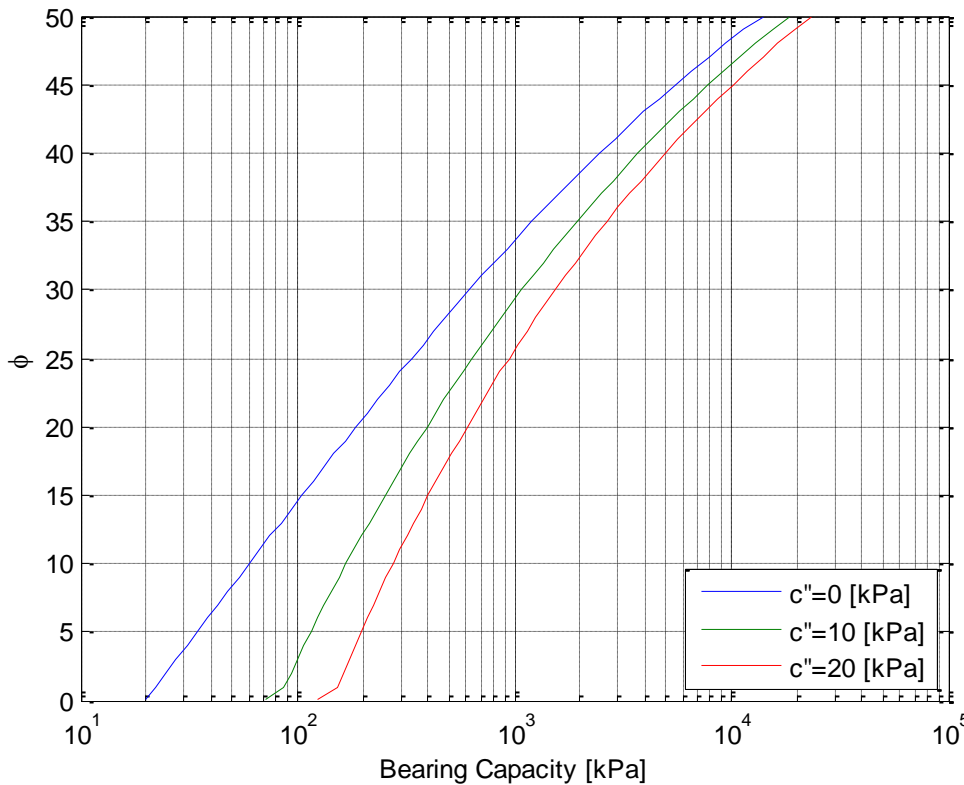
The drained case is evaluated by using the Brinch Hansen's equation. The considered case is for a vertical load acting at the centre of a circle foundation and the bottom side of the foundation is horizontal. Three cases have been evaluated, considering three values of the drained cohesion, in particular 0, 10 and 20 kPa. It has been considered a surcharge at the side of the foundation of 20 kPa, which can simulate the condition of 1 meter surcharge of dry sand.

Table 12 "Drained Bearing capacity in according to the Brinch Hansen Equation for; $c'=0$ kPa, $c'=10$ kPa, $c'=20$ kPa"

phi	Bearing Capacity[kPa]		
	$c'=0$ kPa	$c'=10$ kPa	$c'=20$ kPa
0	20.0	71.4	122.8
5	34.4	114.9	195.4
10	59.8	167.6	275.4
15	104.7	252.6	400.4
20	185.5	394.0	602.5
25	334.9	638.7	942.6
30	622.4	1083.1	1543.9
35	1206.2	1940.2	2674.1
40	2480.4	3725.3	4970.2
45	5542.0	7834.4	10126.9
50	13922.7	18642.4	23362.1



Graph 4 "Factors N_q , N_c , N_γ function of the friction angle ϕ "



Graph 3 "Drained bearing capacity in according to: $c' = 0$ kPa, $c' = 10$ kPa, $c' = 20$ kPa"

3.6 Summary

The aim of this chapter is to show and discuss the theory behind the Brinch Hansen Equation and the Prandtl solution. These equations have been studied, because they are the ones suggested by the Eurocode 7. Regarding the Brinch Hansen equation, it has been shown the dependency of the friction angle with the bearing resistance, Graph 4.

In according to EN 1997-1:2004, special precaution shall be taken when the eccentricity of loading exceeds 0.6 of the radius of the circular footing.

4. Case Studied, Foundation Typologies, Shell Design

This chapter starts with a brief introduction of the geology of the area. This part is not essential but can be considered important to know the typologies of the soil beneath the ground surface. Secondly the case studied with soil investigations can be introduced. Then it can be made an overview of the foundations typologies that can be used for a tank structure. At the end of the chapter the calculation of the tank's shell will be performed in according to API 650 method, to know the dead load of the structure. Basically in this chapter there are not comparisons between the norms, but it describes the case studied and it is a base for the further calculations.

4.1 Geology of the area

The Iraqi area of interest is the region of the city of Basrah, the city is located between Kuwait and Iran. This city is called the Iraq's mouth and it is known by natives as "Venice of East". The location of the city is shown in Figure 7.



Figure 7 "Mesopotamian Region Map"(Google Maps)

The region under exam is called "Mesopotamian Plain". The geology of the area it is useful to predict the stratigraphy of the area, and to pick up a typology of foundation. The Mesopotamian area is a deposition basin.

It is situated along the plate boundaries of the African Plate and the Arabian plate which are in collision. The foreland basin receives sediment that is eroded off the adjacent mountain belt, filling with thick sedimentary successions that thin away from the mountain belt the two main rivers of the area, Tigris and Euphrates.

The lithology of the area range from clays, silts, sands, gravels to boulders, according to depositional environment similar to the Netherlands situation.

4.2 Case Studied

The particular case studied is referred to the design a foundation tank with 8 meters in diameter and 10 meters in height. The liquid stored will be crude oil. The soil investigations of the area have been provided by P.E.G. (Project Europe and Global) company. The input parameters are soil investigation data (SPT) and laboratory tests (Atterberg limits, sieve classification, undrained compressive strength test, consolidation testes, vane test). They have been attached in Appendix C.

The following descriptions of the area have been taken from the site investigation report. The project is located in at Basrah, the site is general a flat area. The soil investigation considered is composed of a borehole advanced by continuous flight auger. Samples have been recover by using (7.5-10 cm) diameter thin walled Shelby tubes, Appendix C. Other samples were also obtained by performing Standard Penetration Test (SPT), taken from split spoon sampler.

4.3 Subsoil stratification

In according EN 1997-1, the depth to be considered for the calculations should be at which the effective vertical stress due to the foundation load is 20% of the effective overburden stress. However it may be also roughly estimated as 1 to 2 times the foundation width. Since the foundation is approximately 10 meters wide, it can be assumed to consider a soil profile of depth not smaller than 20 meters.

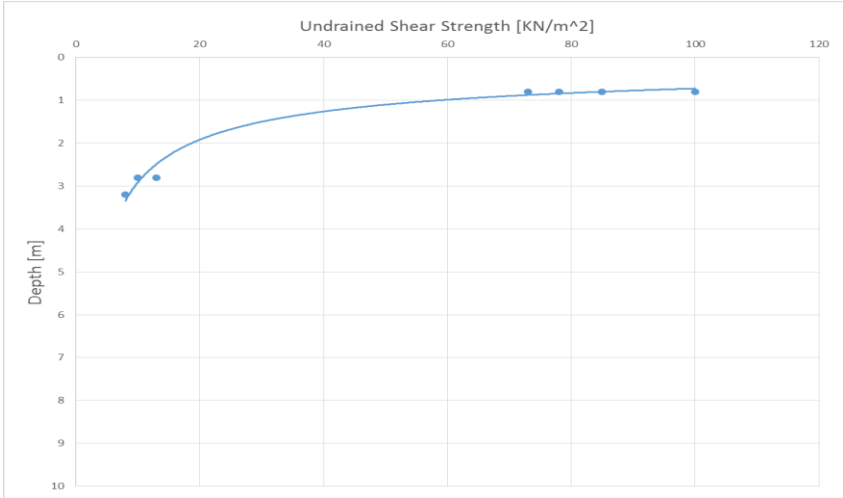
1. The crust soil layer consist of medium dense brown silty sand, the thickness of this layer is about 1.5 m.
2. The second soil layer consists of very soft to soft grey lean clay (CL) which extends to depth ranging around 16 m.
3. Third soil layer consists of medium to very still grey lean clay (CL) or silt with sand which extend to about 25 m.

4.4 Soil strength results

The strength values provided for this thesis have been performed by using the unconfined compressive strength test. There have been also provided the counted number of SPT's blows.

Table 13 "Laboratory Soil Tests, Basrah site"

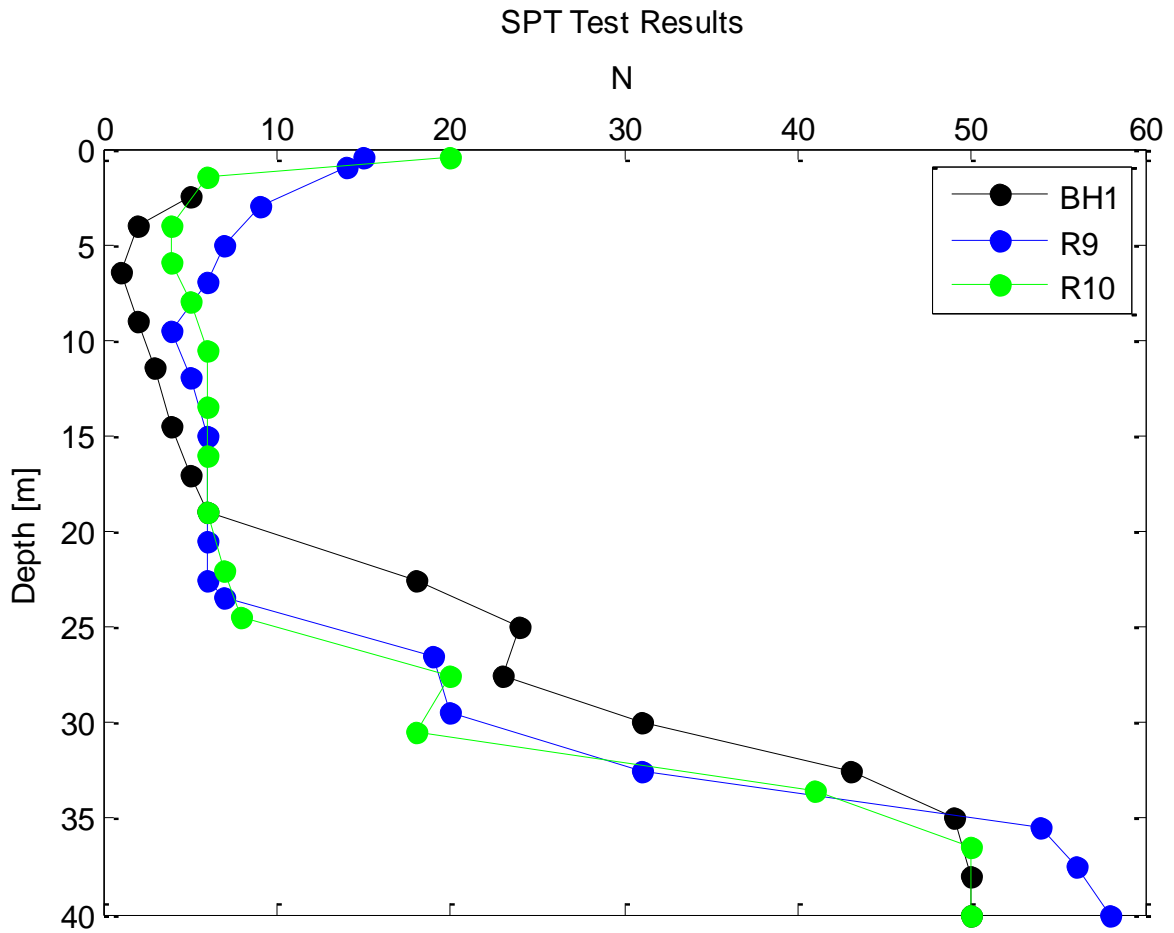
BH NO	Depth [m]	ϕ	Unit Weight [KN/m^3]	
			Wet	Dry
1	24.5-25.0	32	1.96	1.64
	32.0-32.5	33	2.00	1.69
	37.5-38.0	35	2.05	1.77
R9	0.5-1.0	33	1.73	1.60
	32.0-32.5	35	2.02	1.73
	37.0-37.5	36	2.01	1.74
R10	3.5-4.0	31	2.14	1.91
	30.0-30.5	33	1.98	1.66
	36.0-36.5	32	2.01	1.73



Graph 5 "Undrained Shear strength from UCS test, Basrah site"

Regarding the undrained shear strength c_u it has been evaluated by Unconfined Compressive Strength tests (UCS).

The SPT blows versus depth for the boreholes 1, R9 and R10 are reported in the following graph, Graph 6.



Graph 6 "SPT blows data, Basrah site"

Consolidation properties of the soil is shown in the following table.

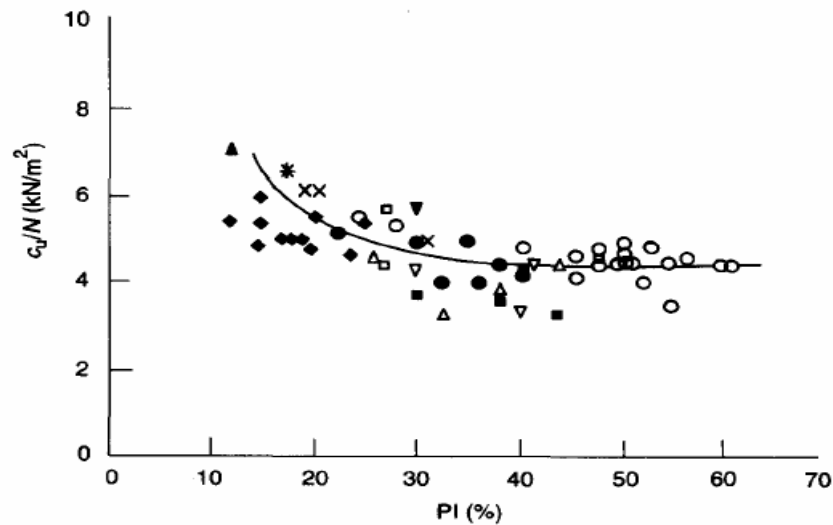
Table 14 "Oedometer test's data, Basrah site"

BH No.	Depth [m]	e_o	C_c	C_r	$P_c [kN/m^2]$	$P_o [kN/m^2]$
1	0.5-1.0	0.74	0.20	0.053	75	20
	3.0-3.3	1.45	0.33	0.066	78	48

Where:

- e_o initial void ratio
- C_c compression index (Normally Consolidate)
- C_r Compression index (Preconsolidation)
- P_c Preconsolidation pressure
- P_o Overburden pressure

In according to (Clayton, Simons, & Matthews, 1982), it is possible to obtain the undrained cohesion c_u from the N blow of an SPT test knowing also the index of plasticity PI. The chart has been developed for a 100 mm diameter specimen



Graph 7"Correlation between SPT N values and undrained shear strength c_u , function of plasticity referred to 100 mm diameter specimens (Clayton, Simons, & Matthews, 1982)"

4.5 Foundation Types

According to PIP STE03020 five common foundation typologies can be chosen: Concrete Ringwall, Crushed Stone Ringwall, Concrete Slab, Compacted Granular filled, Pile Foundation.

In according to (Long & Garner, 2003), foundation type chosen is function of the following parameters:

- Anchorage
- Load and soil bearing capacity
- Heterogeneity of the soil properties
- Tank's diameter

In according to the case studied, crushed stone ringwall and compacted granular fill can be skipped because they do not provide anchorage against seismic load. According to PIP 03020, slab foundation is not economically convenient when it shall be bigger than 6.5 meters.

Then, according to PIP STE 03020, the two more advantageous foundation types are concrete ringwall and pile foundations.

Regarding ringwall foundation it is less expensive than the pile foundation, in this thesis ringwall foundation will be firstly checked if it is suitable for the case studied. In Appendix B, a table with the advantages and disadvantages between the ringwall and pile foundation has been put, it has been taken directly from the PIP 03020.

4.6 Summary

The aim of this chapter is to describe the case studied, in particular the soil profile. In according to the soil properties gained and the size of the structure, possible foundation typologies are a ringwall or pile foundations. Then for the scope of this thesis a ringwall foundation will be studied first and if not feasible mitigations actions or a pile foundation will be designed.

4.7 Shell Design

The design of the shell is performed by using the one-foot method presented in API 650. The calculation of the shell is performed merely to know the dead weight which acts on the foundation. For this reason just the method presented in API 650 will be used. The analysis will be performed to know the thickness of the structural shell and it will not touch the design of the structural details. It will defined also the weight of the roof, by assuming that it has a thickness as the shell course. The method used to perform the ultimate analysis of the shell courses is the ASD method. Regarding structural elements, the method foresees the reduction of the material strength, in particular the yield strength of the steel given by the tables included in the API 650.

4.7.1 Shell

Steel used is A 516M, grade 65, with a tensile strength 450 MPa, and minimum yield strength of 240 MPa. The reduced strength is in according to two design scenario: during the life of the structure and during the hydrostatic test. Generally during the hydrostatic test, it is considered a higher resistance of the shell. The reason why two different strength are considered, is because the hydrostatic test is performed by introducing water inside the tank a leakage or rupture of the tank's shell, will produce a leakage of water, while the scenario will be worst during the life of the tank when a leakage will be by oil. The steel design strength $S_d = 160 \text{ MPa}$ and the hydrostatic stress design is $S_t = 180 \text{ MPa}$. For a tank with a diameter smaller than 15 m, the bottom annular plate shall have a minimum thickness of 6 mm and the upper plates 5 mm. The calculation foresees to determine the minimum thickness as the greater value calculated during hydrostatic test or during the life of the structure. This method is called one-foot method, it is an empirical approach, and it requires that any shell course is simply designed for the pressure 300 mm ($\approx 1 \text{ ft}$) above the edge of the course, rather than the higher pressure at the bottom.

For the following design, in according to a previous design, annular plates of the shell with a width of 2440 mm and a height of 2020 mm, have been used. Since the tank is 10 meters height, the shell will be design for five annular shell plates. The shell course thickness is designed by considering two situations: firstly considering the weight of the product liquid and secondly the test situation when the tank is filled with water, and the corrosion allowance of the shell is not considered and how aforementioned, the yield strength of the shell's material is increased.

$$t_d = \frac{4.9D(H - 0.3)G}{S_d} + CA \quad \text{Equation 4.1}$$

$$t_t = \frac{4.9D(H - 0.3)}{S_t} \quad \text{Equation 4.2}$$

Where:

- D tank's diameter, m
- H tank's height, m
- G specific gravity of the product liquid
- CA corrosion allowance $\text{min} = 3 \text{ mm}$

The following table summarise the results of the analysis.

Table 15 "Minimum Thickness of the Shell courses"

No		Width [mm]	Depth [mm]	t.design [mm]	t.hydrostatic [mm]	tsd [mm]	Weight [Kg]
1	A516 GR. 65N	2440	10000	5.37	2.10	6	2304
2	A516 GR. 65N	2440	8080	4.90	1.69	5	2020
3	A516 GR. 65N	2440	6060	4.41	1.25	5	2020
4	A516 GR. 65N	2440	4040	3.91	0.81	5	2020
5	A516 GR. 65N	2440	2020	3.42	0.37	5	2020
Tot. Shell Weigh							10382

4.7.2 Annular Bottom Plate

In according to Table 15, from a stress calculation, the bottom annular shell course should be 6 mm thick, however for this particular annular plate other limitations are required. The bottom annular plate, should have a minimum thickness of 6 mm plus 3 mm considering corrosion. Therefore the bottom plate should have at least 9 mm of thickness.

4.7.3 Bottom Plate

Before calculating the annular plate thickness and width, it is required to work out the maximum stresses in the first shell course during product and hydrostatic test.

$$\sigma_{test} = \frac{4.9D(H - 0.3)}{t_{sd}} = 50.3MPa \quad \text{Equation 4.3}$$

$$\sigma_{prod} = \frac{\frac{4.9D(H - 0.3G)}{t_{sd}}}{1 - \frac{CA}{t_d}} = 101.3MPa \quad \text{Equation 4.4}$$

In according to (API 650, section 5.5.3, table 5-1a), with the following level of stresses in the first shell course and the thickness of the first shell plate, the minimum thickness of the bottom plate is 6 mm. Considering a corroded allowance of 6 mm, the design thickness of the bottom plate is 12 mm.

This plate shall have a radial width that provides at least 600 mm between the inside bottom of the shell and any welded bottom plate. The outside radial width should be greater than 50 mm and can be also calculated by the following formula:

$$L_{out} = \frac{215t_b}{(HG)^{0.5}} = 449.2 \text{ mm} \quad \text{Equation 4.5}$$

Where:

- t_b thickness of the annular bottom plate [mm]
- H tank height [m]
- G specific gravity of the product liquid

However it cannot be less then $0.035D = 140 \text{ mm}$. The final external width will be 450 mm.

Then the bottom plate will be 12 mm thick and it will be 1050 mm width.

4.8 Summary

To summarise the shell has the following geometrical dimensions:

Table 16"Summary of the Shell thickness sections"

No	Depth [mm]	tsd [mm]
5	0-2020	5.00
4	2020-4040	5.00
3	4040-6060	5.00
2	6060-8080	5.00
1	8080-10000	9.00
Bottom Plate		9
Annular Bottom Plate		12

Once the shell has been dimensioned, it is possible to work out the dead vertical load. Considering the weight of the shell, bottom plate and bottom annulus, the dead load of the shell is 17216 *Kg*. For the project has been also considered a fixed roof, which, from a previous project can be assumed to have a weight of 2700 *kg*. From the data of the dead load is possible to work out the seismic response of the structure (Zravkov, 2010).

5. Wind and Seismic action Determination

A comparison of the wind force determination is conducted between the ASCE 7-10, (reference standard for wind load of API 650) and the Eurocode EN 1991-1-4:2004 method. It has been also used the ISO 4354 (2009) which is a unified norm, used generally in war scenario to make a conversion between the standards of different countries. The two methods use the same basic theory, based on gust factor approach (Bashor, Kareem, & Moran, 2009). The two methods have been compared in term of the resultant force worked out.

Regarding seismic response, the chapter has been divided two parts; firstly the methodologies have been compared, secondly there has been a comparison of the response spectrum, where the main differences are encountered.

5.1 Eurocode Wind Load

In according to the Eurocode wind load determination is performed by following different steps, the whole calculations are shown in Appendix D. In this part the main equations are shown.

In according to the Eurocode EN 1991-1-4:2004, the basic wind velocity is calculated from the characteristic 10 minutes wind velocity of the area. The value of this reference velocity, for the city of Basrah, is difficult to be founded. However it is easier to find the three second gust wind velocity which is the characteristic wind velocity used by the American standard (UFC, 2013), and by the use of ISO 4354 it is possible to convert into the ten minutes wind velocity $V_{ref T=3 s} = 1.46 V_{ref T=600 s}$.

$$V_{ref T=600 s} = v_{b,0} = 35.2 \text{ m/s} \quad \text{Equation 5.1}$$

The basic wind velocity is calculated as:

$$v_b = c_{dir} c_{season} v_{b,0} \quad \text{Equation 5.2}$$

Where:

- v_b is the basic wind velocity defined at 10 meters above ground
- $v_{b,0}$ fundamental value of the 10 minutes basic wind velocity
- c_{dir} is the directional factor, it is generally founded in nation annexes, but the value of 1.0 is generally suggested
- c_{season} is the season factor, it is generally founded in nation annexes, but the value of 1.0 is generally suggested

The peak velocity pressure can be calculated as:

$$q_p(z) = c_e(z) q_b \quad \text{Equation 5.3}$$

Where:

- $c_e(z)$ is the exposure factor for flat terrain the value can be taken from a chart:
- q_b is the basic velocity pressure $= \frac{1}{2} \rho v_b^2$
 - ρ it is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storm the recommended value is 1.25 kg/m^3

A remark should be done regarding the peak velocity pressure $q_p(z)$ Equation 5.3, this equation will be used to calculate the internal and the external peak pressure. The equation is function of z , the elevation point until where the peak pressure shall be calculated. For the particular case studied where the height of the tank considered is greater than the base dimension, in according to Figure 8 the structure may be considered, being composed of two parts: a lower part extending upward from the ground by an height equal to the diameter, and an upper part consisting of the remainder. The pressure load distribution is considered rectangular, the resultant force carried out from the analysis will be applied at the centre of the pressure distribution scheme.

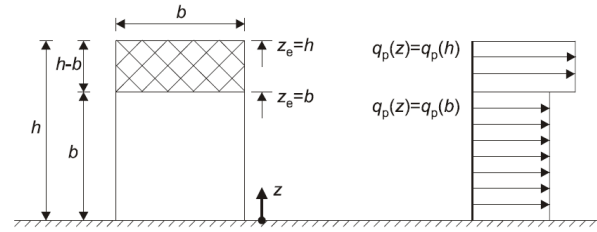


Figure 8 "Wind velocity pressure profile EN 1991-1-4:2004"

From the method presented in the Eurocode, it is possible to calculate the internal and external pressure generated by the wind. The external wind pressure w_e :

$$w_e = q_p(z_e)c_{pe} \quad \text{Equation 5.4}$$

Where

- c_{pe} is the external pressure coefficient

While internal pressure w_i

$$w_i = q_p(z_i)c_{pi} \quad \text{Equation 5.5}$$

Where

- c_{pi} is the internal pressure coefficient

The internal and external pressure have been calculated respect the two part for which the structure has been divided. The calculation of the internal pressure is important when the structure is empty, when the roof is not applied yet.

The wind force acting on the structure is:

$$F_w = c_s c_d c_f q_p(z) A_{ref} \quad \text{Equation 5.6}$$

Where:

- The coefficient $c_s c_f$ can be considered equal 1 for structures lower than 15 meters, this is similar to the peak factor that is used in the ASCE 7-10 method.
- c_f is the force coefficient it is function of the roughness of the shell surface, in the present project it has been worked out the value for a cast iron and a rush surface.

$$c_{f,0} = 1.2 + \frac{0.18 \log\left(10 \frac{k}{b}\right)}{1 + 0.4 \log\left(\frac{Re}{10^6}\right)} \quad \text{Equation 5.7}$$

- k is the equivalent roughness factor, 0.2 and 2, for cast iron and rust shell's surface.

$$c_f = c_{f,0} \Psi_\lambda \quad \text{Equation 5.8}$$

- A_{ref} is the reference area which is the section area perpendicular to the wind direction.

5.2 API 650 Wind Load

API 650, uses a straightforward equation to calculate the design wind pressure, however this equation is for a particular case:

- three second gust wind velocity is 190km/h
- terrain category is of type C.

For more general cases the reference is to use the ASCE 7-10:2005.

The method used is called Main Wind-Force Resisting System (MWFRS).

This method foresees to calculate; firstly the wind velocity pressure and secondly transform it into design wind pressure. The wind pressure velocity is calculated from the gust velocity which is defined in according to the risk category of the building and the location of the building. By using following coefficients: wind directional, topographic, velocity exposure importance factor and velocity pressure is calculated the wind velocity pressure q_z is worked out.

$$q_z = 0.613 K_z K_{zt} K_d V^2 I \quad \text{Equation 5.9}$$

Where:

- K_z velocity pressure exposure coefficient, 1.04
- K_{zt} topographic factor, 1
- K_d wind directional factor, 0.95
- V basic wind speed $51,4 [m/s]$
- I importance factor 1.15

From the value of the wind velocity pressure, the design wind pressure p is calculated in the general form as:

$$p = qG C_p - q_i (G C_{pi}) \quad \text{Equation 5.10}$$

Where:

- G gust factor
- C_p external pressure coefficient
- $G C_{pi}$ internal pressure coefficient, for enclosed buildings is ± 0.18 the sign is if it is acting forward or away from the structure.
- $q = q_z$ for windward walls evaluated at height z above the ground
- $q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h (maximum building height)
- $q_i = q_h$ for windward walls, side wall, leeward walls and roofs of enclosed buildings and for negative internal pressures
- $q_i = q_z$ where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure (z is the free height above the storage liquid level).

It can be suggested to calculate the value of q_i at the height H , the total height of the tank, because this is the case when the tank is empty and the maximum internal pressure is generated.

The horizontal force is defined as:

$$F_h = q_h(GC_r)A_f \quad \text{Equation 5.11}$$

- q_h is the horizontal velocity pressure, calculated at height h
- A_f vertical projected area of the rooftop structure or equipment on a plane normal to the direction of the wind = Bh
 - B is the horizontal dimension of the building
 - h is the mean roof height
- (GC_r) for roof top structures and equipment, it can be reduced linearly from the value 1.9 to 1.0 as the value of A_f is increased from $(0.1Bh)$ to (Bh)

The vertical uplift force is:

$$F_v = q_h(GC_r)A_r \quad \text{Equation 5.12}$$

- q_h is the velocity pressure of the equation above, calculated at height h
- A_r horizontal project area of the rooftop structure or equipment
- (GC_r) 1.5 for A_r less than $(0.1BL)$ shall be permitted to be reduced linearly from 1.5 to 1.0 as the value of A_r is increased from $0.1BL$ to BL , where B is the horizontal dimension of the building and L is the horizontal dimension of the building

5.3 Methods Overview

Both norms are in agreement with the 10 meters reference height from where the basic wind velocity is calculated, they are also in agreement to the 2% annual probability exceedence equivalent to a mean return period of fifty years.

However the reference time velocity changes, in particular, Eurocode considers as fundamental value of wind velocity, the characteristic ten minutes wind velocity. ASCE 7-10:2005, considers the three second gust wind velocity. Since the difficulties to find the characteristic wind velocity for the EN 1991 for the city of Basrah, it is possible to use ISO 4354, where relationship between the two values can be used:

$$V_{ref\ T=3\ s} = 1.46 V_{ref\ T=600\ s} \quad \text{Equation 5.13}$$

For both the codes the velocity pressure is function of the following coefficients:

$$q = \frac{1}{2} \rho V_0^2 C_{exposure} C_{terrain} C_{direction} C_{importance} C_{other} \quad \text{Equation 5.14}$$

Where:

- ρ air density $1.25\ kg/m^3$
- V_0 basic wind velocity
- $C_{exposure}$ velocity profile or exposure coefficient
- $C_{direction}$ directionally factor
- $C_{importance}$ building importance factor
- C_{other} takes in to account other aspects such as hurricane zone, shielding or surface roughness.

A difference from the standards is the coefficient C_{other} , for EN 1991-4-1 it is not encounter while for ASCE 7-10 this term is relative to special wind conditions such as hurricanes.

A similarity between the standards is how the along wind load is worked out. It is calculated by multiplying the wind pressure by the influent area of the section (the influent area is the horizontal cross section of the building on the plane orthogonal to the wind direction) and by other factors. The difference however is that the velocity pressure, for ASCE 7-10:2005, has a uniform distribution along the structural section while for the Eurocode it is function of the height. Once the velocity pressure is worked out the wind pressure p is calculated in the same manner for both the codes:

$$p = qC_p \tag{Equation 5.15}$$

Where:

- q velocity pressure
- C_p pressure coefficient

Pressure coefficient accounts for the roughness of the terrain. With reference to the Eurocode the terrain roughness can be divided in five categories, while ASCE 7-10, divides the terrain in three categories.

The whole calculations are summarized in appendix E. The theoretical approaches used to calculate the wind velocity pressure acting along the shell are similar, differences in magnitude of the forces acting are observed. In the Table 17 the wind resultant actions; for a 10 meters height and 8 meters diameter tank are summarized. It is possible to see that the wind pressure for the ASCE 7-10 is calculated for the bottom and upper part of the structure; where for lower part is meant the section from the ground level until a distance equal to the diameter while the upper part is the remaining part. It is possible to see that the method of ASCE 7-10, has a higher estimation in magnitude of the wind velocity pressure; ASCE 7-10 considers a wind velocity pressure higher than EN 1991-1-4:2004. With regards to the force resultant, the calculated wind force of ASCE 7-10 is 20% lower than the one calculated in according to EN 1991-1-4:2004.

Table 17 "Wind Loads EN 1991 characteristic forces, and ASCE 7-10 design forces (8 meters diameter 10 meters height)"

		EN 1991-1-4:2004	ASCE 7-10	
Velocity Pressure [Kg/ms ²]	lower tank part	1,627	1,541	5%
	upper part	1,813	1,541	15%
Resultant Force [N]		102,788	123,350	-20%
Uplift Load [N]		-	7,944	

The reason why the wind force is lower for the EN 1991-1-4:2004 method, is because the wind force is founded by multiplying the wind velocity pressure by the effective area and a force factor. The force factor accounts for the roughness of the surface, in according to EN 1991-1-4:2004, it is equal to 0.77 as aforementioned explained. In according to ASCE 7 the factor is called GC_r , as aforementioned explained and it is equal to 1. An important difference between the norms is that the wind force for the Eurocode, in every wind load combination, is lowered than 40% as is possible to see in the combination factor of Table 4.

5.4 Seismic Action

For the seismic action, the calculation is performed using the API 650 and the EN 1998-4:2006 methods. The approaches used by the two standards are similar, differences are when the spectrum of response is calculated. The dynamic analysis of a liquid storage tank is a problem involving fluid-structure interaction. When a tank containing liquid starts to vibrate, the liquid exerts on the tank wall and on the base, impulsive and convective hydrodynamic pressure in addition to the hydrostatic pressure. The impulsive part corresponds to the lower liquid part in the tank and behaves like a mass rigidly connected to the shell, in the upper part of the tank the liquid mass has a sloshing movement and exerts convective hydrodynamic pressure. Generally a common approach is by using a double single degree of freedom system one which is rigidly connected to the tank shell (impulsive) and the second which is attached to the tank through a springs (convective), see Figure 10. By using this method is possible to model the interaction between the liquid and the wall of the tank. The model then tends to divide the entire liquid mass into two part, i.e. impulsive (m_i) and convective mass (m_c). A qualitative representation of impulsive and convective hydrodynamic distribution is represented in Figure 10. The convective and impulsive masses are at a distance from the bottom plate h_c (or h_c^*) and h_i (or h_i^*). The difference between h_c and h_c^* and h_i and h_i^* is that; the ones marked with the asterisk takes into account the effect of the development of the hydrodynamic pressure at the base of the tank, while the other no, Figure 10. Both the models which considers the spring mass model requires to obtain m_i , m_c , h_i , h_i^* , h_c , h_c^* and K_c in according to Figure 10. The impulsive and the convective masses moves with an impulsive and convective period of vibration T_i and T_c .

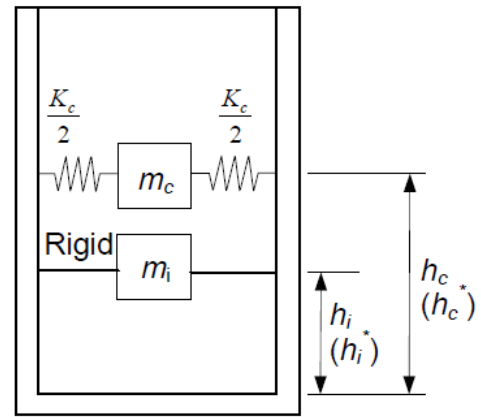


Figure 10 "Liquid-filled tank modelled: double single-degree-of-freedom system (Malhotra, Wenk, & Wieland, 2000)"

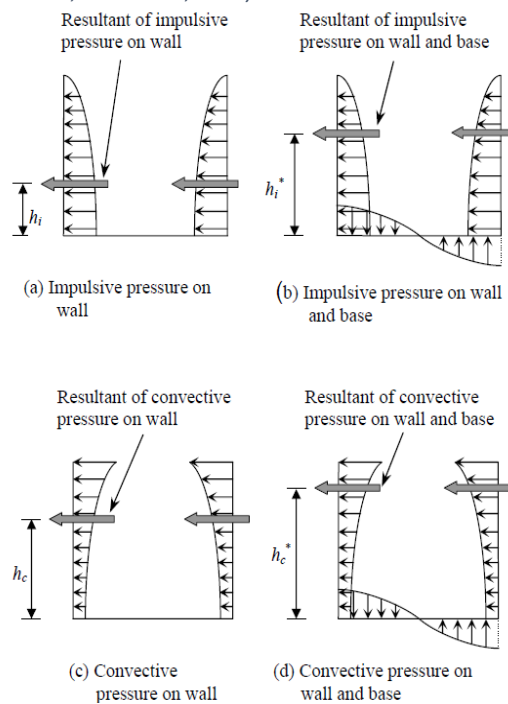


Figure 10 "Qualitative description of hydrodynamic pressure distribution on tank wall and base"

5.5 Differences and Similarities

The time period of the impulsive mode (T_i) of vibration is equal in both the norms:

$$T_i = C_i \frac{h\sqrt{\rho}}{\sqrt{t} \sqrt{D} \sqrt{E}}$$

Equation 5.16

Where:

- C_i coefficient of time period for impulsive mode.
- h maximum depth of the liquid
- D inner diameter of circular tank

- t thickness of the shell
- E modulus of elasticity of the shell
- ρ mass density of the liquid

The impulsive time coefficient period is equal for both the codes and it is:

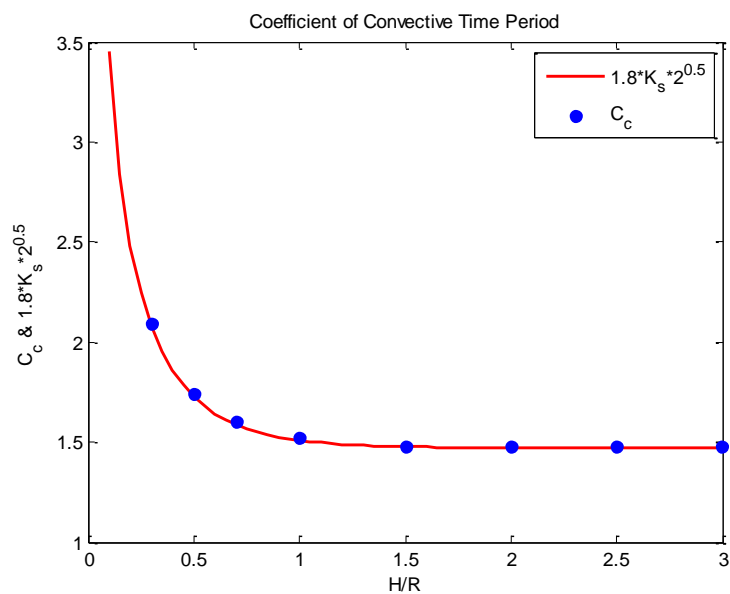
$$C_i = \left(\frac{1}{\sqrt{\frac{h}{D} \left(0.46 - \frac{0.3h}{D} + 0.067 \left(\frac{h}{D} \right)^2 \right)}} \right) \quad \text{Equation 5.17}$$

Regarding the time coefficient of the first convective mode period, the expressions in the two standards are different, in particular:

Table 18 "Convective time equations, accordingly to API 650 and EN 1998-4:2006"

API 650	EC 1998-4:2006
$T_c = 1.8 K_s \sqrt{D} = 1.8 K_s \sqrt{2} \sqrt{R}$ <p style="text-align: right;">Equation 5.18</p>	$T_c = C_c \sqrt{R}$ <p style="text-align: right;">Equation 5.20</p>
$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68h}{D}\right)}}$ <p style="text-align: right;">Equation 5.19</p>	C_c it is founded from a table

It is possible to compare in a graph for different values of h/D the values of C_c used by the Eurocode and $1.8 K_s \sqrt{2}$ (corresponding C_c coefficient of the API 650).



Graph 8 "Comparison of convective coefficient time period API 650 and EN 1998"

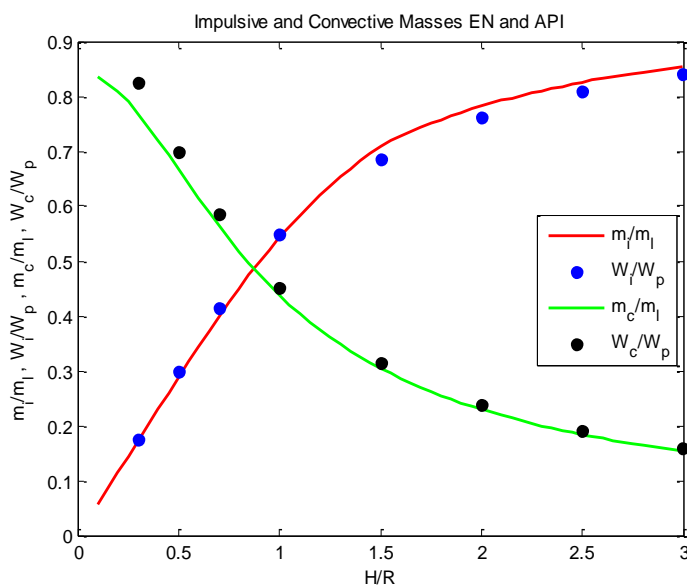
From Graph 8, is possible to see that the two coefficients of convective time period worked out by using the two norms are similar in values.

The effective impulsive and the convective masses calculated in according to API 650 and EN 1998-4:2 are given in Table 19.

Table 19 "Impulsive and Convective Mass equations, according to API 650 a EN 1998"

Impulsive Mass	
API 650	EN 1998-4-1
When: $\frac{D}{h} \geq 1.333$ $W_i = \frac{\tanh\left(\frac{0.866D}{h}\right)}{\frac{0.866D}{h}} W_p$ Equation 5.21 $\frac{D}{h} < 1.33$ $W_i = \left[1.0 - 0.218 \frac{D}{h}\right] W_p$ Equation 5.22 Where: W_p is the product weight	The value of W_i is founded from table
Convective Mass	
API 650	EN 1998-4-1
$W_c = 0.230 \frac{D}{h} \tanh\left(\frac{3.67h}{D}\right) W_p$ Equation 5.23	The value of W_c is founded from table

In the following graph values of the convective and impulsive mass, taken from Table 19, are compared: the values from the Eurocode 8 are represented by dots while the curve are in according to API 650.



Graph 9 "Impulsive and Convective mass equations, according to Eurocode and API W_i, W_p, W_c are referred to API while m_i, m_l, m_c are referred to Eurocode"

From Graph 9 is shown that between the impulsive and convective masses defined from API and Eurocode the values are similar.

5.6 Centre of action of the effective lateral forces.

For API 650 and EN 1998, two equations can be used to calculate the centre of the action for the impulsive and convective masses. They are referred to the case that the foundation is a concrete slab or a ringwall. The centroid for the convective component takes into account the effect of the development of the hydrodynamic pressure at the base of the tank while impulsive mass (the one without asterisk) do not consider this effect see Figure 10.

Table 20 "Impulsive Mass Height equations for ringwall and slab foundation case in according to API 650 and EN 1998"

Impulsive mass height	
API 650	EN 1998-4-1
When: $\frac{D}{h} \geq 1.333$ $h_i = 0.375 h$ Equation 5.24	The value of h_i is founded from table
$\frac{D}{h} < 1.33$ $h_i = \left[0.5 - \frac{0.094D}{h} \right] h$ Equation 5.25	
API 650	EN 1998-4-1
When: $\frac{D}{h} \geq 1.333$ $h_i^* = 0.375 \left[1.0 + 1.333 \left(\frac{0.866D}{\tanh\left(\frac{0.866D}{h}\right)} - 1 \right) \right] h$ Equation 5.26	The value of h_i^* is founded from table
$\frac{D}{h} < 1.33$ $h_i^* = \left[0.5 + \frac{0.06D}{h} \right] h$ Equation 5.27	

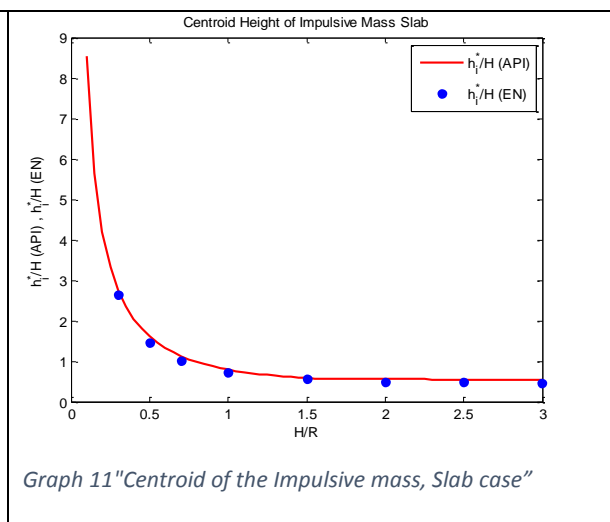
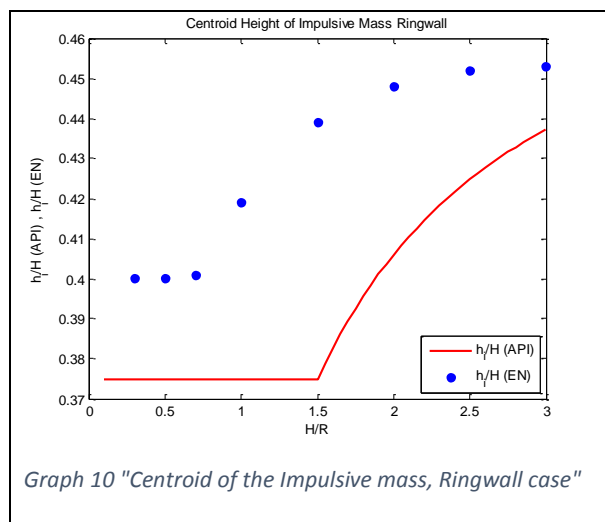
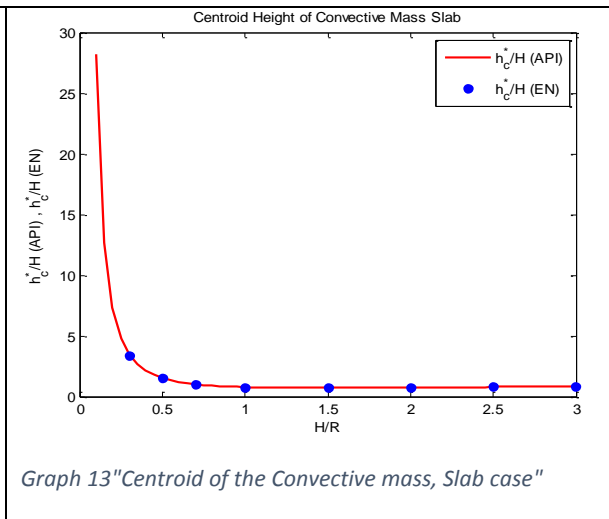
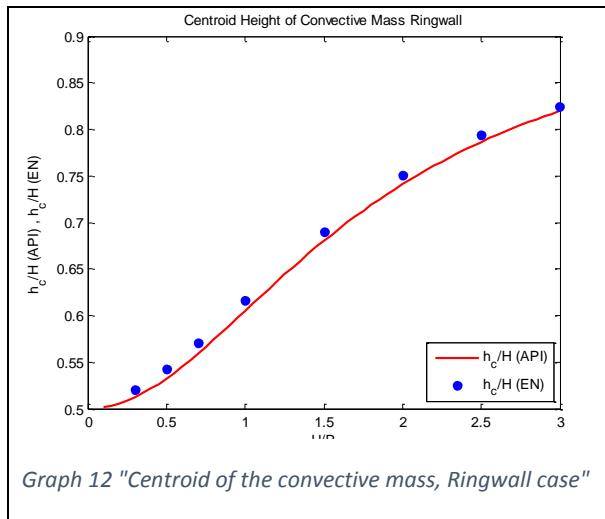


Table 21 "Convective Mass Height equations for ringwall and slab foundation case in according to API 650 and EN 1998"

Convective mass height	
API 650	EN 1998-4-1
$h_c = \left[1.0 - \frac{\left(\cosh\left(\frac{3.67h}{D}\right) - 1 \right)}{\frac{3.67h}{D} \sinh\left(\frac{3.67h}{D}\right)} \right] h$ <p style="text-align: right;">Equation 5.28</p>	The value of h_c is founded from table
API 650	EN 1998-4-1
$h_c^* = \left[1.0 - \frac{\cosh\left(\frac{3.67h}{D}\right) - 1.937}{\frac{3.67h}{D} \sinh\left(\frac{3.67h}{D}\right)} \right] h$ <p style="text-align: right;">Equation 5.29</p>	The value of h_c^* is founded from table



Regarding the centroid of the impulsive and convective masses, the results worked out from the API and European code, are similar. However the impulsive centroid mass for Eurocode is 5% higher than the API's one.

5.7 Overturning Moment

As it was aforementioned, the moment acting at the base of the foundation is calculated in according to the respective moment arms h_i, h_i^*, h_c, h_c^* . The arms marked with asterisk will be used to design the slab foundation while without asterisk is used to design ringwall. The two norms differentiate each other in terms how they combine the convective and the impulsive effects, in particular API 650 uses the Square Root of the Sum of the Square while the EN 1998-4:2006 uses the algebraic sum rule to combine the effects.

Table 22 "Overturning Seismic Moment equations, API 650 and EN 1998"

API 650	EN 1998-4:2008
$M = \sqrt{[A_i(W_i h_i + W_w h_w + W_r h_r)]^2 + [A_c(W_c h_c)]^2}$ <p style="text-align: center;">Equation 5.30</p>	$M = (m_i h_i + m_w h_w + m_r h_r) S_e(T_{imp}) + m_c h_c S_e(T_{con})$ <p style="text-align: center;">Equation 5.31</p>
$M^* = \sqrt{[A_i(W_i h_i^* + W_w h_w + W_r h_r)]^2 + [A_c(W_c h_c^*)]^2}$ <p style="text-align: center;">Equation 5.32</p>	$M^* = (m_i h_i^* + m_w h_w + m_r h_r) S_e(T_{imp}) + m_c h_c^* S_e(T_{con})$ <p style="text-align: center;">Equation 5.33</p>

5.8 Acceleration Coefficient

The design acceleration coefficients of the two norms, are derived from different spectrum of response. The first difference between the acceleration spectrum of response, is related to the peak ground acceleration, which is the input parameter.

In according to EN 1998-4-1 it is suggested to consider the design ground acceleration with 10% probability of exceedence in 50 years for a ground profile A. Ground profile A is related to rock-like geological formation. For API 650, the values of the PGA (peak ground acceleration), defined in ASCE 7-10, is referred for a ground class B (which as for the Eurocode) rock formation with a 2% probability of exceedence in 50 years.

Eurocode acceleration parameter is reported in Appendix D.

In according to (UFC, 2013), values of PGA for the city of Basrah can be taken from (UFC, 2013).

- PGA peak ground acceleration for the city of Basrah = 39%g
- S_s mapped, maximum considered earthquake, 5% damping, spectral response at short period (0.2 seconds) = 103%g
- S_1 mapped, maximum considered earthquake, 5% damping, spectral response at a period of 1 second = 49%g

Then the input values provided for the two norms are mismatched and cannot be mixed up together.

5.8.1 Design Response spectrum Eurocode

The design spectrum of response of the Eurocode is presented in this paragraph. For horizontal component of the seismic action the design spectrum, $S_d(T)$ shall be defined by the following expressions:

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \quad \text{Equation 5.34}$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \quad \text{Equation 5.35}$$

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases} \quad \text{Equation 5.36}$$

$$T_D \leq T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \quad \text{Equation 5.37}$$

Where:

- a_g design value from the peak value
- β is the lower bound factor for the horizontal design spectrum
- q behaviour factor
- S soil factor
- S_d design spectrum
- T_B is the lower limit of the period of the constant spectral acceleration branch
- T_C is the upper limit of the period of the constant spectral acceleration branch
- T_D is the value defining the beginning of the constant displacement response range of the spectrum

The soil factor is a function of the ground class. For the case studied, ground profile belong to category D, the relative soil factor S is 1.35.

The behaviour factor is a reduction factor, which reduces the force obtained from a linear analysis in order to account the nonlinear response of the structure. The behaviour factor is associated with energy dissipation of the structure. For Eurocode it is possible to perform an elastic analysis and through the behaviour factor, develop the "design spectrum". The damping coefficient assumes different values in according to different structures, however for tank design the reference norm is EN 1998-4:2006. The suggested values are:

- $q = 1$ always when the convective response (sloshing) is evaluated

Using the value of $q = 1$ it means that the response is considered with damping equal zero (elastic).

Considering the Impulsive part:

- $q = 2$ for unanchored tanks
- $q = 2.5$ for anchored tank, but considering that the anchors are ductility designed, allowing an increase in anchor length without rupture equal to $R/200$ (R is the radius of the tank)

Then according to EN 1998-4:2006, then the global equilibrium should be checked during the seismic design.

Regarding the time period factors, they are function of the soil category and geology of the area. In according to the Ground type D and the values T_B, T_C, T_D are 0.2, 0.8 and 2 seconds respectively.

5.8.2 Design Response Spectrum API 650

API 650, uses a response spectrum based on 5% damping. The response spectrum is defined as:

$$0 \leq T \leq T_S \quad A_d(T) = 2.5 K Q F_a P G A \frac{I}{R_w} \quad \text{Equation 5.38}$$

$$T_S < T \leq T_L \quad A_d(T) = 2.5 K Q F_a P G A \frac{T_S}{T} \frac{I}{R_w} \quad \text{Equation 5.39}$$

$$T > T_L \quad A_d(T) = 2.5 K Q F_a P G A \frac{T_S T_L}{T^2} \frac{I}{R_w} \quad \text{Equation 5.40}$$

Where:

- F_a Acceleration base site coefficient (at 0.2 sec. period)
- K Coefficient to adjust the spectral acceleration from 5% to 0.5% damping
- $P G A$ Peak ground acceleration coefficient, in according to ASCE 7-10.
- Q Scaling factor for countries outside USA is equal 1
- R_w Force reduction coefficient using allowable stress design method
- S_s Mapped maximum considered earthquake, 5% damped, spectral response acceleration parameter at short period (0.2 sec)
- T_L Regional dependent transition period ground motion, equal to 4 sec (API 650, p.E-11).
- T_S Upper limit of the period of the constant spectral acceleration branch 1.27 sec
 - $T_S = \frac{F_v S_1}{F_a S_s}$
 - F_v Velocity base coefficient This factor is function of the site geological class of the area, (table E2, API650, p. E-7)
 - F_a Acceleration-based site coefficient (table E1, API650, p. E-7)

The spectrum response of the API 650 is based on assuming 5% damped response for the impulsive mode and 0.5% damped spectra for the convective mode. K is the coefficient to adjust the spectrum from 5% to 0.5% damping. Since the soil factors are based on 5% damping when we uses $K = 1$ we are describing the impulsive response, while for the convective $K = 1.5$.

R_w is the force reduction coefficient of API 650 provides table for its value. Four cases can be considered two different values are for the convective and impulsive response, moreover for any of the response typology there are two cases, in particular it is evaluated the case when the tank is self or mechanically anchored. The following table summarizes the values.

Table 23 " Reduction Force coefficients, ringwall and slab foundation, API 650"

Anchorage system	Rw_i	R_wc
Self-Anchored		3.5
Mechanically-Anchored		4

In according to the case studied, the tank does not satisfy the requirements for the unanchored situation, then anchors should be provided.

5.9 Seismic Foundation design API 650

In according to API 650, foundation should be proportioned to resist peak anchor uplift and overturning bearing pressure. In case of a ringwall foundation is used, the soil over the footing can be used to resist the foundation uplift. When the seismic analysis is performed the wind load is not considered.

In according to API 650, when the bearing pressure is evaluated the product pressure over the foundation shall be multiplied by a factor $(1 + 0.4A_v)$ and the foundation should be designed to resist the eccentric loads with or without the vertical seismic acceleration, where A_v is the vertical acceleration coefficient.

It should be also checked the overturning of the foundation this formula is used when anchored tanks are considered:

$$\frac{0.5D(W_p + W_f + W_T + W_{fd} + W_g)}{M_s} \geq 2.0 \quad \text{Equation 5.41}$$

Where:

- W_p Weight of the product load
- W_f Weight of the tank bottom
- W_T Total weight of the tank shell, roof, framing , knuckles, product, attachment, appurtenances, snow load,
- W_{fd} Total weight of the tank foundation
- W_g Weight of soil directly over tank foundation footing
- M_{slab} Slab Moment

This mechanism assumes that the tank rigidly rotate and the rotation occurs around the toe.

5.10 Seismic Foundation design Eurocode

For Eurocode the relative geotechnical seismic code is EN 1998-5:2004, in according to the section for footing, the ultimate state design criteria shall be checked against failure bearing capacity. In according to EN 1998-4:2006, it should be checked also the overturning.

Bearing capacity is calculated by the following expression:

$$\frac{(1 - e\bar{F})^{c_T} (\beta\bar{V})^{c_T}}{(\bar{N})^a [(1 - m\bar{F}^k)^{k'} - \bar{N}]^b} + \frac{(1 - f\bar{F})^{c'_M} (\gamma\bar{M})^{c_M}}{(\bar{N})^c [(1 - m\bar{F}^k)^{k'} - \bar{N}]^d} - 1 \leq 0 \quad \text{Equation 5.42}$$

Where:

$$\bar{N} = \frac{\gamma_{Rd} N_{Ed}}{N_{max}}$$

Equation 5.43

$$\bar{V} = \frac{\gamma_{Rd} V_{Ed}}{N_{max}}$$

Equation 5.44

$$\bar{M} = \frac{\gamma_{Rd} M_{Ed}}{BN_{max}}$$

Equation 5.45

- B is the foundation width
- \bar{F} is the dimensionless inertia force for purely cohesive soil it is:
 - $\bar{F} = \frac{\rho a_g S B}{\bar{c}}$
 - ρ unit mass of the soil
 - S soil factor

- N_{max} is the ultimate bearing capacity of the foundation under a vertical centred load, for purely cohesive soil is:
 - $N_{max} = (\pi + 2) \frac{\bar{c}}{\gamma_M} B$
 \bar{c} is the undrained shear strength of soil, c_u , for cohesive soil, or the cyclic undrained shear strength τ_{cy} for cohesionless soils
 γ_M is the partial factor for material properties taken differently from the material factors used in the static analysis
- γ_{Rd} is the model partial factor, defined as:
 - 1.00 for medium-dense to dense sand
 - 1.15 loose dry sand
 - 1.50 loose saturated sand
 - 1.00 non sensitive clay
 - 1.15 sensitive clay
- $a, b, c, d, e, f, m, k, k', c_T, c'_M, \beta, \gamma$ are numerical parameters the values can be taken from the following table

Table 24 "Coefficient of seismic bearing capacity check, Equation 5.42 EN 1998-4:2006"

	Purely cohesive soil	Purely cohesionless soil
a	0.70	0.92
b	1.29	1.25
c	2.14	0.92
d	1.81	1.25
e	0.21	0.41
f	0.44	0.32
m	0.21	0.96
k	1.22	1.00
k'	1.00	0.39
c_T	2.00	1.14
c_M	2.00	1.01
c'_M	1.00	1.01
β	2.57	2.90
γ	1.85	2.80

Eurocode 8, considers mandatory to have an assessment of the site of construction, which shall be carried out to ensure that hazards of liquefaction or highly densification susceptibility in event of an earthquake are minimized.

5.11 Freeboard

Freeboard calculation is related to the sloshing effect, basically the calculation is done to calculate the freeboard that should be guaranteed to avoid spill of the product liquid when seismic sloshing will occur.

5.12 Comparison

A real comparison between the two spectrums of seismic response, can be done if different tank structures are considered, in this thesis just a case has been considered.

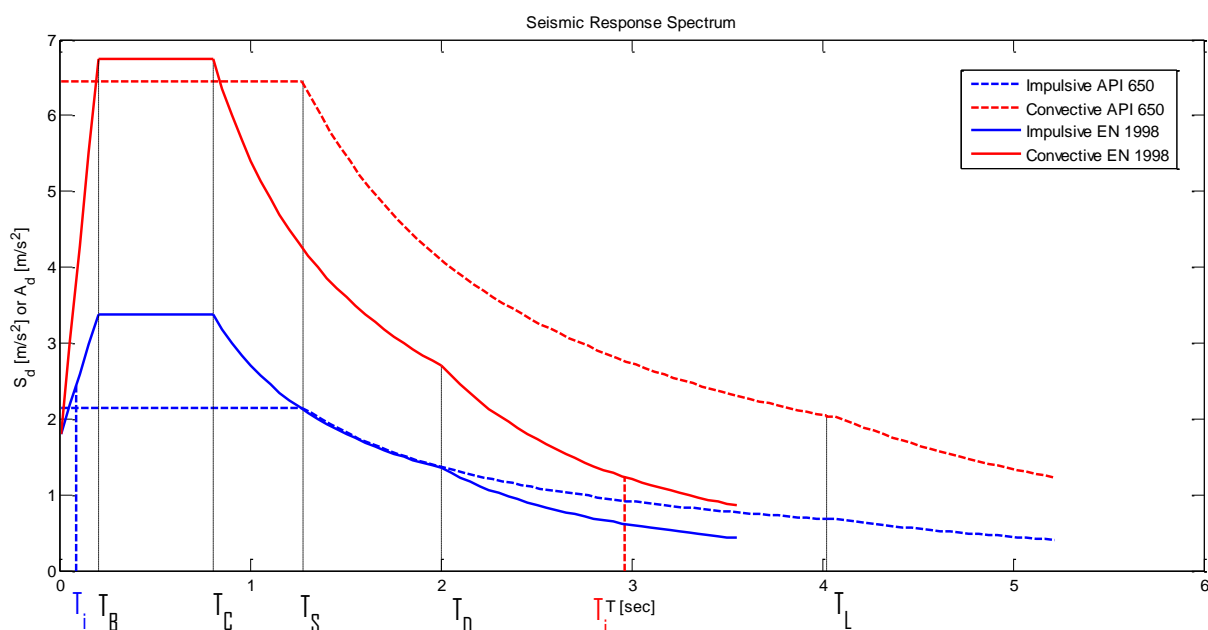
However difference between the norm's spectrums can be noticed:

1. Regarding the convective response, the API spectrum consider a spectrum with 0.5% of damping while EN 1998 considers an elastic response (damping equal 0%), as shown in Graph 14. It is possible to see that the horizontal branch of the API spectrum has a lower value of the acceleration coefficient respect to the Eurocode's method.
2. Such as the convective acceleration, API considers a higher damping for the impulsive response. The horizontal branch of the API spectrum is located in correspondence of a lower value of the acceleration response.
3. API 650 considers the end of the period with constant spectral acceleration longer, $T_s > T_C$ in according to Graph 14. For Eurocode the dissipation of energy starts earlier.

The seismic response spectrums of the two norms, for the specific case studied, have been reported in Graph 14. It is referred to a tank based on ringwall foundation and mechanically anchored. The soil profile considered is of category D for API 650 and EN 1998.

Table 25 " Impulsive and Convective acceleration coefficient, for the case studied, according to API 650 and EN 1998"

	API 650	EN 1998-4:2008
Impulsive	$A_i = 2.45 \text{ m/s}^2$	$S_I = 2.44 \text{ m/s}^2$
Convective	$A_c = 2.78 \text{ m/s}^2$	$S_C = 1.23 \text{ m/s}^2$



Graph 14" Impulsive and Convective Response spectrum, for the case studied, according to API 650 and EN 1998"

Regarding the moments calculated in according to the two norms they have been summarized in Table 26.

Table 26 "Case Studied: Overturning Seismic Moment API 650 and EN 1998"

API 650	EN 1998-4:2008
$M_{ringwall} = 2.31 \cdot 10^6 \text{ Nm}$	$M_{ringwall} = 2.96 \cdot 10^6 \text{ Nm}$
$M_{slab}^* = 2.91 \cdot 10^6 \text{ Nm}$	$M_{slab} = 3.21 \cdot 10^6 \text{ Nm}$

For the case studied, despite the convective acceleration coefficient of the Eurocode is almost half of the American one (Table 25), the destabilizing moment founded using the Eurocode is higher than the one of API 650 Table 26.

Two reasons can explain this controversial point between the acceleration components and the convective and impulsive moments:

- The convective mass is a small component respect to all the masses involved in the analysis
- API method uses the square root of the sum of the square (SRSS) of the separate components to calculate the destabilizing moment, while for Eurocode is considered the algebraic sum Table 22. The SRSS sum reduces the magnitude of the total response.

Such as shown in Graph 14 as much as the impulsive time increases bigger the “Eurocode” impulsive acceleration response of the system is. Similarly the “API” convective response increases as the convective time increases. The impulsive and the convective time are functions of the height and the diameter of the tank, Equation 5.16 and Equation 5.18.

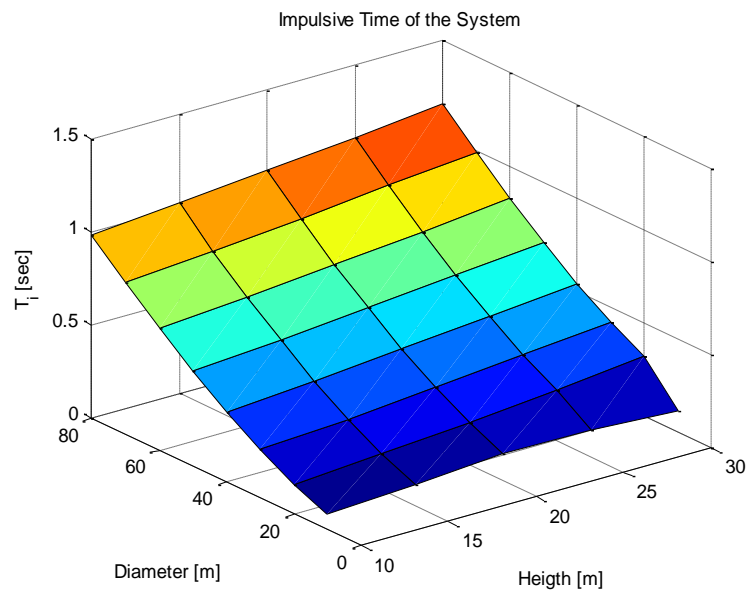
5.12.1 Impulsive and Convective time H/D dependency

From Equation 5.46 and Equation 5.47 before defined (as Equation 5.16 and Equation 5.17), impulsive time is defined as:

$$T_i = C_i \frac{h\sqrt{\rho}}{\sqrt{\frac{t}{D}}\sqrt{E}} \quad \text{Equation 5.46}$$

$$C_i = \left(\frac{1}{\sqrt{\frac{h}{D} \left(0.46 - \frac{0.3h}{D} + 0.067 \left(\frac{h}{D} \right)^2 \right)}} \right) \quad \text{Equation 5.47}$$

It is possible to know the dependency of the impulsive time T_i with D (tank’s diameter) and H (tank’s height) by plotting the values of T_i in function of D and H as shown in Graph 15. It is possible to see that for constant values of the diameter, the increment of the tank’s height does not give a big increment of the T_i as much as the diameter is increased.



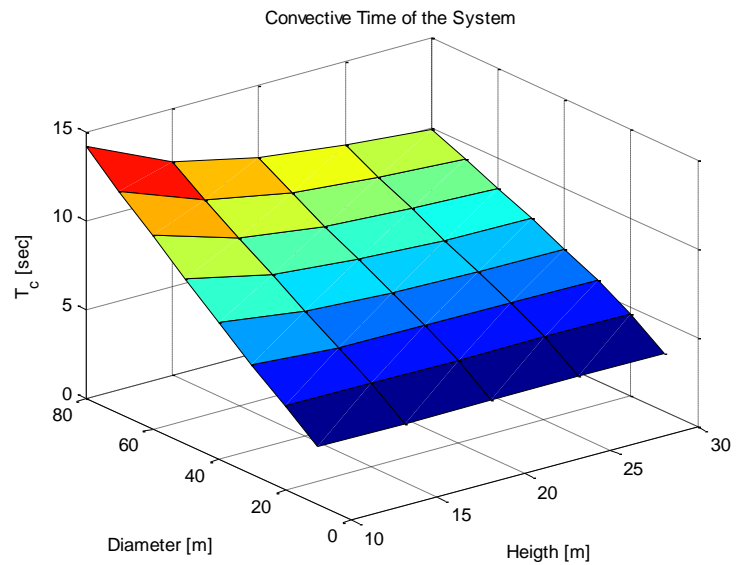
Graph 15 "Time of the maximum impulsive response, function of the tank's Height Diameter ratio"

Maximum convective time response can be worked out from Equation 5.48 and Equation 5.49 previously defined (as Equation 5.18 and Equation 5.19).

$$T_c = 1.8 K_s \sqrt{D} = 1.8 K_s \sqrt{2} \sqrt{R} \quad \text{Equation 5.48}$$

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68h}{D}\right)}} \quad \text{Equation 5.49}$$

The convective time response is plotted in function of D and H in Graph 16. It is possible to see that T_c increases as D and H increase. It can be suggested that there is a higher dependency with the tank's diameter rather than with the tank's height. This higher dependency can be justified by the physical meaning of T_c , that represents the sloshing time of the content. Basically it is travelling time of the waves on the tank, then bigger is the diameter, bigger is the distance which has to be covered and higher is the time required. From Graph 16, when the height of the tank is reduced, keeping constant the diameter, there is an increment in T_c with H .



Graph 16 "Time of the maximum convective response, function of the tank's Height Diameter ratio"

6. Foundation Design

In this paragraph load combinations and dimensioning of the foundation will be treated. In the first part, combination loads of the Eurocode and API 650 will be evaluated. Then, the foundation section will be dimensioned. It will be performed a settlement analysis and consolidation analysis considering the use of surcharge and drains. Bearing capacity of the foundation will be analysed accordingly to the global equilibrium failure mode (Brinch-Hansen equation).

6.1 Load Combinations

Load combinations have been performed in according to the Eurocode and the API 650, attached in Appendix F and Appendix G respectively. For the Eurocode seven load cases scenario have to be perform and each of them should be performed twice, in according to two load combination equations (Equation 6.1 & Equation 6.2). The expressions were already introduced in chapter 2.2.5.1 (Equation 2.14 & Equation 2.15).

$$\left\{ \begin{array}{l} \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \\ \sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \end{array} \right. \quad \begin{array}{l} \text{Equation 6.1} \\ \text{Equation 6.2} \end{array}$$

Combination factors ψ and ξ are taken from Table 27:

Table 27 "Combination Factors for Equation 6.1 and Equation 6.2, where ψ and ξ are referred to characteristic loads" EN 1991-1:2003

Short Title	Design Situation/leading variable action	Permanent action		Accompanying variable action 1 (main)		Accompanying variable action 2		Accompanying variable action 3,4 etc	
		Description	ξ_1	Description	$\psi_{0.1}$	Description	$\psi_{0.2}$	Description	$\psi_{0.3}$ $\psi_{0.4}$ etc.
D	Liquid Discharge	Self Weight	0.9	Liquid filling	1.0	Foundation settlement	0.7	Snow, wind, thermal	0.6
								Imposed loads, imposed deformation	0.7
I	Imposed load or deformation	Self Weight	0.9	Liquid Filling	1.0	Imposed Deformation	0.7	Snow, wind, thermal	0.6
								Imposed loads	0.7
S	Snow	Self Weight	0.9	Liquid Filling	1.0	Snow	0.6	Imposed loads	0.7
WF	Wind and full	Self Weight	0.9	Liquid Filling, full tank	1.0	Wind	0.6	Imposed loads	0.7
WE	Wind and empty	Self Weight	0.9	Liquid, empty tank	0.0	Wind	0.6	Imposed loads	0.7

Such is possible to see from Table 27, the difference between Equation 6.1 and Equation 6.2 are that, in Equation 6.1 permanent loads $G_{k,j}$ are unfactored and characteristic principal variable actions $Q_{k,1}$ is factored, vice versa in Equation 6.2.

Regarding API 650, load combination foresees to perform eight load combination analysis Table 9.

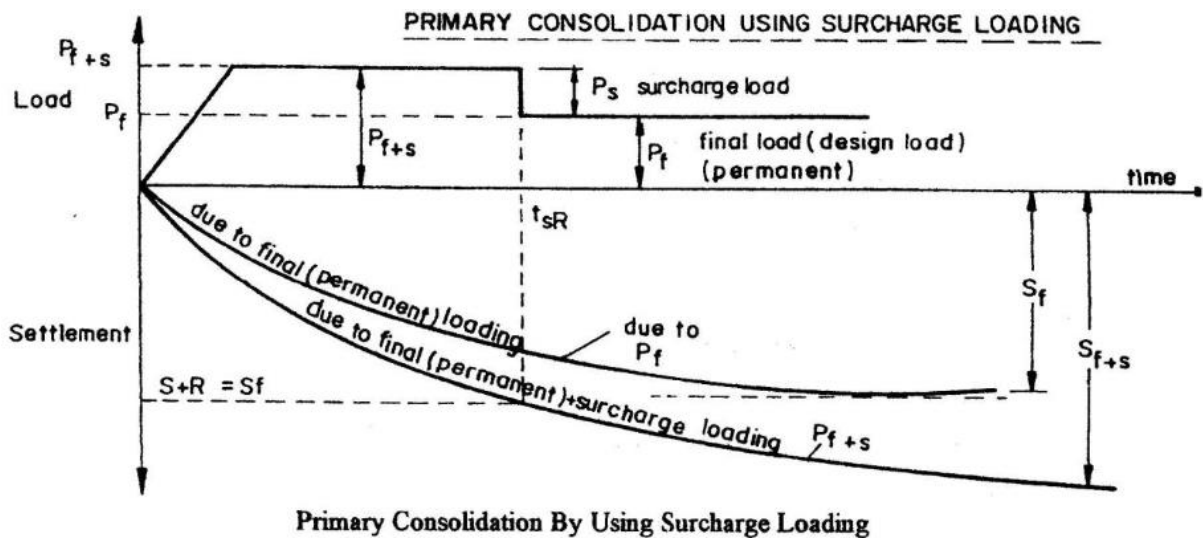
6.2 Ground Improvement

Soil beneath the foundation has a low permeability and a low compressibility, it could be suggested that bearing stability analysis may be controlled by the undrained condition. Soil layer is composed of soft clay, from laboratory tests the undrained bearing capacity is $5kPa$. From a preliminary bearing capacity analysis it has been seen that an increment of the undrained cohesion to a final value of $40kPa$, will satisfy the undrained equilibrium for all load cases. In according to (Terzaghi, Peck, &

Mesri, 1996), undrained cohesion is dependent from effective stress then a way to increase the effective stresses may be, by using preload.

6.2.1 Preload

To accelerate consolidation and to anticipate settlements it is often used a preloading technique. The main idea is as shown in Graph 17.



Graph 17 "Preload consolidation process"

Basically with the preload, greater load level (surcharge) is placed over the soil, settlements are forced to be completed in a shorter time than would have been occurred under design loads. Surcharge load results also to increase the undrained shear strength of the soil, because as it is stated in (Chin, 2005) the undrained cohesion is function of the effective vertical stress. In according to (Terzaghi, Peck, & Mesri, 1996), a method presented by (Mesri, 1988), correlate the increment of the vertical loads $\Delta\sigma'_{vc}$ with the undrained shear strength Δs_u . For soft clays it is possible to use the following expression:

$$\Delta s_u = 0.22\Delta\sigma'_v \quad \text{Equation 6.3}$$

The preload should be wide enough to cover all the area which can be affected by the mechanism of failure stability.

Preloading soft materials with low permeability, generally takes a lot of time to reach consolidation, a method to reduce the time is by using preload in combination with vertical drains.

6.2.1.1 Preload Geometry

Preload geometry should take into account the depth and the width of the possible bearing failure surface. Defining D_e as the "equivalent" thickness equal to: diameter of the tank plus half of the thickness of the granular soil. For (Duncan, ASCE, D'orazio, & ASCE, 1984) failure mechanism relative to global equilibrium is shown in Figure 11. The depth of the failure mechanism is

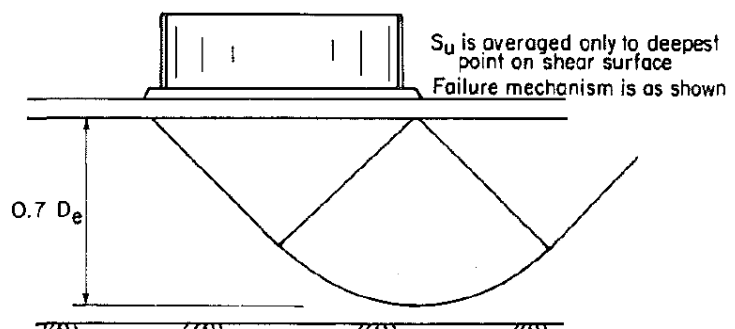


Figure 11 "Undrained Failure Mechanism referred to the case of Global Equilibrium (Duncan, ASCE, D'orazio, & ASCE, 1984)"

positioned at $0.7 D_e$. Considering the foundation diameter and the thickness of the granular soil of 1.5 meters the depth of the failure surface will be positioned approximately at around 9 meters from the ground surface. In according to Equation 6.3 to have a gain in the undrained cohesion of $35kPa$, the increment of the preconsolidation pressure shall be of around $160 kPa$. The stress increment in the soil can be calculated by using Boussinesq theory to validate the increment of the effective preconsolidation stress and consequently the gain of the undrained cohesion. Regarding the width of the overburden, in according to Figure 11, from the tank edge the failure mechanism can be considered symmetrical, so it can be suggested to use an overburden embankment at least 10 meters wide form the tank's edge.

6.2.1.2 Vertical Stress Calculation

The Boussinesq method is based on the following hypotheses:

- The tensions are transferred to the soil through a flexible membrane, it means that the load applied gives and uniform distribution of the pressure on the soil surface (q) as shown in Figure 13.
- The deformation in the soil for the load applied does not generate a changing on the distribution of the load applied.

The changing of the stresses within the soil mass $\Delta\sigma_v$ can be expressed by the following equation:

$$\Delta\sigma_v = qI_s \quad \text{Equation 6.4}$$

- q load pressure applied by the footing, net load pressure, difference between the load applied and the initial applied stress
- I_s influence factor

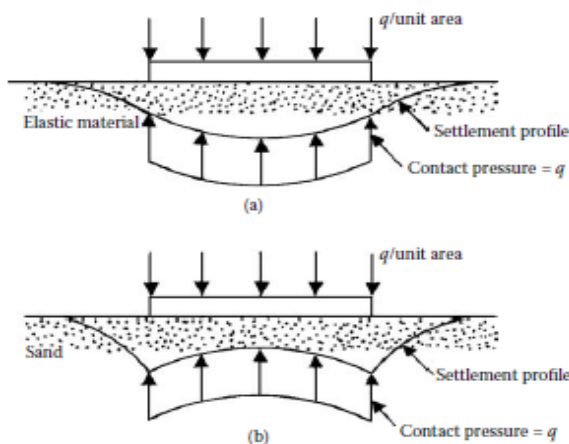


Figure 13 "Contact pressures and settlements for flexible foundation: (a) elastic material; (b) granular soil" (Braja, 2009)

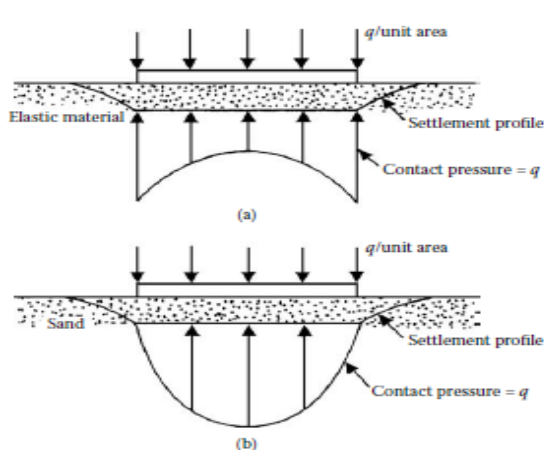


Figure 13 "Contact pressure and settlements for rigid foundation: (a) elastic material; (b) granular soil" (Braja, 2009)

6.2.1.2.1 Embankment's Vertical Stresses

Vertical stresses will be calculated in two points: below the centre of the embankment and five meters far from the centre. If the centre of the embankment will be positioned in correspondence of the tank's centre, these two points will correspond to the centre and to the edge of the tank. Considering the load applied by the embankment as the sum of a uniform and a linearly increasing load. With reference to the stress distribution diagrams shown in Figure 14 the corresponding equations for uniform increasing pressures is:

$$\sigma_z = \frac{q}{\pi} \{ \alpha + \sin \alpha \cos(\alpha + 2\beta) \}$$

Equation 6.5

and for linearly increasing pressure:

$$\sigma_z = \frac{q}{\pi} \left\{ \frac{x}{B} \alpha - \frac{1}{2} \sin 2\beta \right\}$$

Equation 6.6

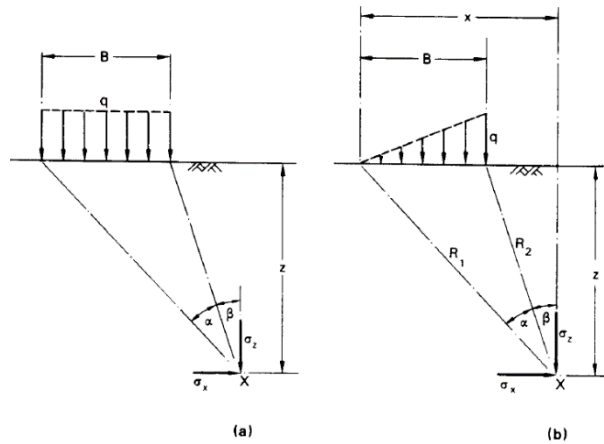


Figure 14 "Model scheme for stress distribution due to (a) uniform pressure and (b) linearly increasing pressure," (Craig, 2004, p. 149)

Assuming the depth of 9 m (7.7 m plus the thickness of the sand of 1.5 m) from the ground surface, where the bottom of the failure surface it may be occur, Figure 11, the increment of the effective stresses due to the surcharge are calculated and shown in Table 28. It is possible to see that to have an increment of the preconsolidation stress of 160 kPa at 9 m of depth, the surcharge should be of 170 kPa; with this preload in correspondence the centre of the embankment there will be 0.92 m of settlements.

By having a surcharge of 170 kPa, using sand to build the preload embankment with a density of 20 kN/m³ the height of the embankment should be 8.5 m. To achieve the final undrained cohesion it is required to consolidate the soil. The time to reach the final consolidation can be calculated by using Terzaghi's theory.

Table 28 "Final Undrained Cohesion due to Surcharge, case studied"

Preload Overburden 170 [kPa]					
Central Point			Tank's Edge		
z (Ground Surface)	Increment of effective stress	Final Undrained Cohesion	z (Ground Surface)	Increment of effective stress	Final Undrained Cohesion
[m]	[kPa]	[kPa]	[m]	[kPa]	[kPa]
0.25	170	42	0.25	170	42
1	170	42	1	170	42
1.5	170	42	1.5	170	42
2	170	42	2	170	42
2.5	170	42	2.5	170	42
3.5	170	42	3.5	169	42
5	169	42	5	168	42
7	167	42	7	166	42
9	164	41	9	163	41
11.5	159	40	11.5	158	40
14.5	152	38	14.5	150	38
18	142	36	18	141	36

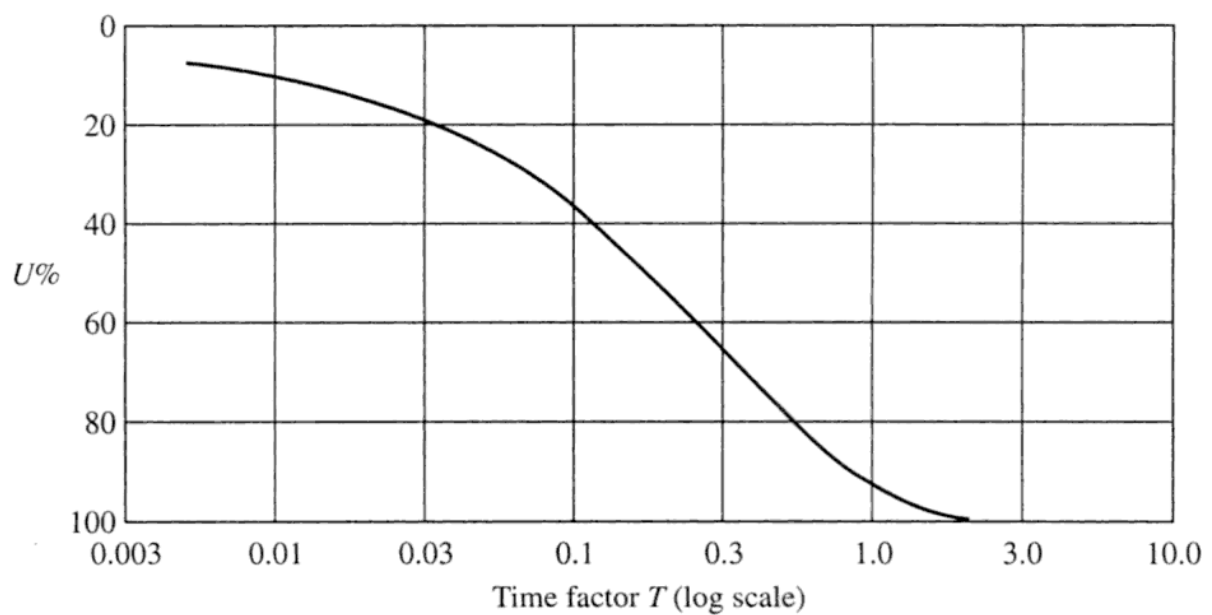
6.2.2 Time of Consolidation

Terzaghi gives an equation to assess the consolidation time is according to the degree of consolidation. For the one dimensional consolidation theory:

$$T_v = \frac{c_v t}{H^2} \quad \text{Equation 6.7}$$

- c_v consolidation coefficient
- H filtration path, if the soil layer considered is drained on two sides, H is half of the layer's thickness 12.5 m
- t consolidation time considered
- T_v time factor

Graph 18 "Relationship between average degree of consolidation and time factor layer drained in both sides (Craig, 2004)"



Time factor for end of consolidation is equal 2 (Graph 18), regarding the vertical consolidation coefficient c_v , it was not provided by soil investigations, then a guess of it should have been done.

6.2.2.1 Coefficient of vertical consolidation

In the soil data it is not reported the coefficient of vertical consolidation, this index tell us the rate at which the compression of the soil takes place, it has the units of m^2/s . It is possible to back calculate its value, from the plasticity index (Sridharan & Nagaraj, 2012). However the back calculation of this index should account of the following aspects, it is directly depended to the hydraulic conductivity and to the consolidation effective vertical stress. The determination of c_v is generally really difficult, even when soil samples are gained, because different values can be obtained from specimens same boring sample (Coduto, Yeung, & Kitch, 2011). The value of c_v generally varies between the ranges of 10^{-7} to 10^{-8} decreasing at the increment of the plasticity index. To have more conservative values of the consolidation index, it is suggested to use higher values of IP in according to the soil data gained. Using Figure 15, the values of c_v have been determined by the use of normal

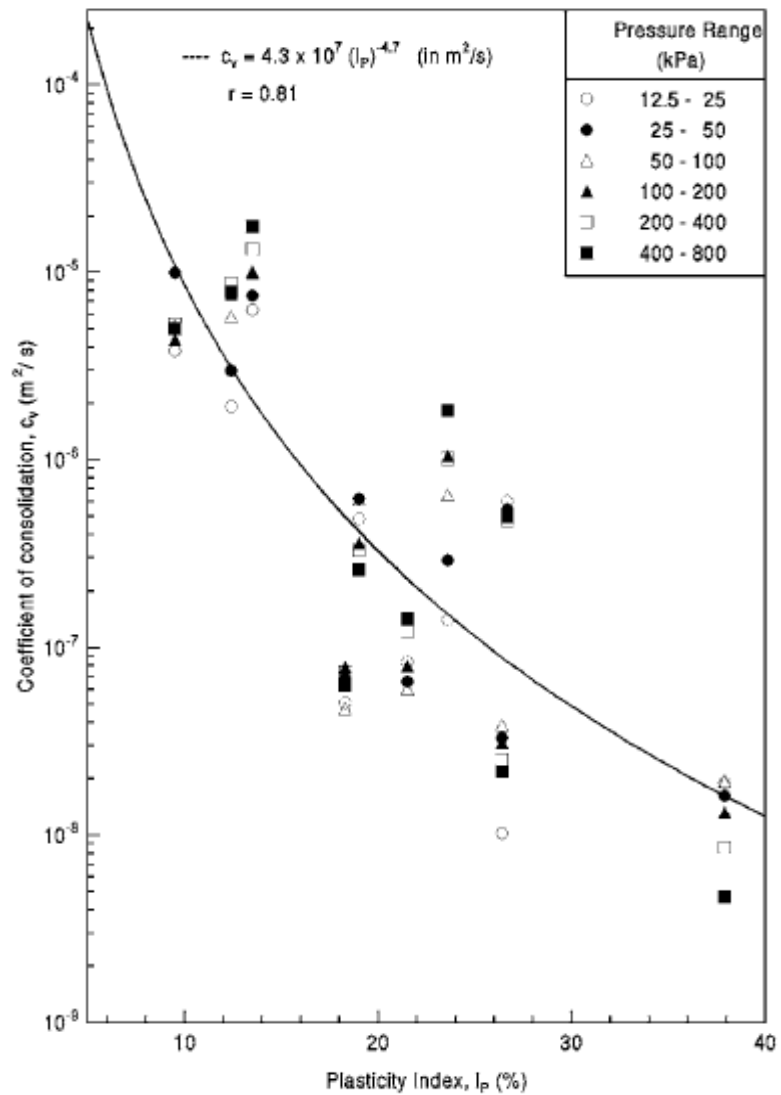


Figure 15 "Relationship between the coefficient of consolidation and the plasticity index for different effective vertical consolidation pressures (Sridharan & Nagaraj, 2012)"

consolidate sample, because they are referred to remoulded samples. However over consolidate samples give higher c_v values (Coduto, Yeung, & Kitch, 2011). Indeed during an oedometer test, the value of c_v decreases during the recompression of the sample, until reaching the normal consolidation stress state, where the increment of the vertical stress leads its value to remain constant. To back calculate the consolidation coefficient from the soil investigation data, it may be better to take the ones corresponding to a depth where the soil is normally consolidate.

In according to Figure 15, considering for the case studied a plasticity index of 20% then $c_v = 2 \cdot 10^{-7} m^2/s$.

By using Equation 6.7 the consolidation time required is 49.5 years.

6.2.3 Vertical Drains

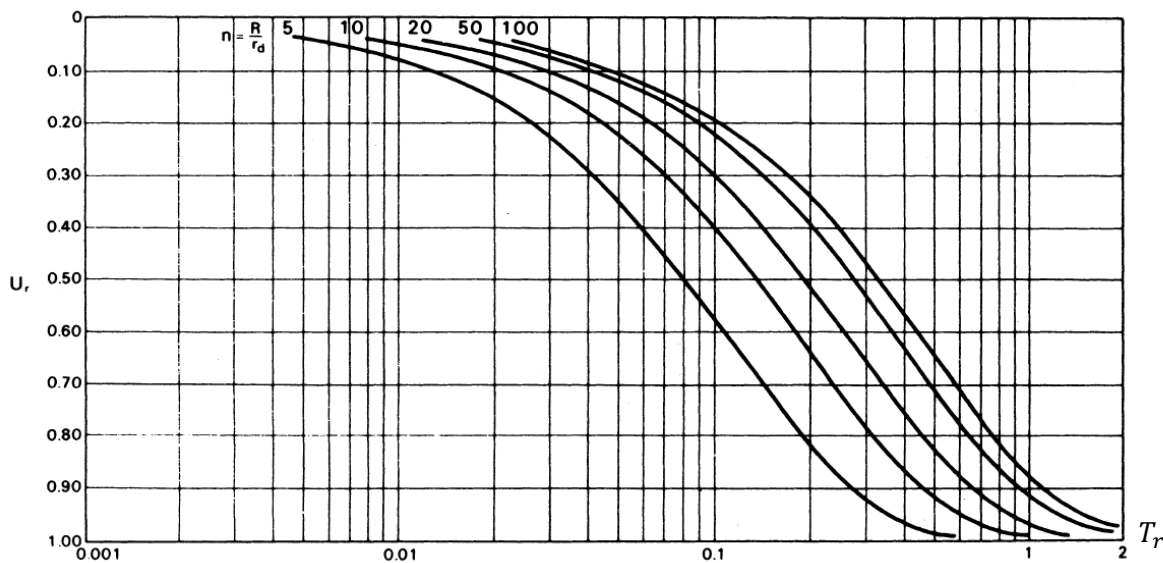
Vertical drains can be used in a triangular or square configuration. Drain are used basically because they accelerate the primary consolidation time.

Drains consist of a central plastic drain core, with holes to allow the drainage and an external filter of geotextile. The model of consolidation with drains can be described by considering an equivalent cylinder of soil of radius R_{eq} with external impermeable surfaces. The equivalent radius R_{eq} assumes different values if the drains have been arranged in a square or triangular scheme.

$$R_{eq} = 0.564s \quad \text{Equation 6.8}$$

- s distance between the drains; 1.5 m

The radial consolidation for equivalent cylinder of soil which drains to the centre has been given by Barron and is based on Terzaghi's theory, solutions for radial consolidation are given in Graph 19. Different solutions are function of the ratio $n = R_{eq}/r$, where r is the drain radius. The n factor worked out is equal to 15.



Graph 19 "Horizontal Consolidation rate, for different n ratio (Craig, 2004)"

It is possible to work out the pore water pressure by the total degree of consolidation U_T which is defined as:

$$U_T = 1 - (1 - U_V)(1 - U_r) \quad \text{Equation 6.9}$$

- U_V average vertical degree of consolidation
- U_r average radial degree of consolidation
- U_T average total degree of consolidation

In accordance with U_V , it is the values worked out by the Terzaghi's theory in accordance with Graph 8, while the average radial degree of consolidation U_r can be worked out from Graph 19 function of the time factor for the radial consolidation T_r . From Equation 6.9, it is possible to see that when the average radial consolidation term is ($U_r = 100\%$, end of radial consolidation), then the second term becomes 0 and the total consolidation degree is equal 100% (total consolidation completed).

$$T_r = \frac{c_h t}{4R_{eq}^2} \quad \text{Equation 6.10}$$

- c_h horizontal coefficient of consolidation

The values of c_h was not provided by soil data, it has been back calculated, in particular, according to (Craig, 2004), the ratio between c_v and c_h is generally between 1 and 2. In this case it has been used a factor equal 2. With $c_v = 4 \cdot 10^{-7} \text{ m}^2/\text{s}$ the time to complete the consolidation is equal to 82.8 days.

t [years]	0.10	0.20	0.30	0.40	1.00	4.00
T_v	0.00	0.01	0.01	0.02	0.04	0.16
U_v	9.00%	10.00%	23.00%	25.00%	37.00%	60.00%
T_h	0.44	0.88	1.00	1.30	1.30	1.30
U_h	85.00%	95.00%	100.00%	100.00%	100.00%	100.00%
U_T	86.35%	95.50%	100.00%	100.00%	100.00%	100.00%
Sett [m]	<u>0.82</u>	<u>0.91</u>	<u>0.96</u>	<u>0.96</u>	<u>0.96</u>	<u>0.96</u>

Table 29 " Total consolidation rates (U_T) in time (t), with vertical drains, case studied"

However is always suggested to check if the consolidation process is ended by using piezometer.

6.3 Foundation Sizing

Foundation sizing has been performed by using a method which is used by an American guideline PIP STE 03020. This design method is widely used also by other guidelines, the basic idea is to size the foundation ringwall so that the soil pressure under the ringwall equals the net bearing capacity of the soil. The minimum foundation width can be determined by using Equation 6.11.

$$b = \frac{P_t + W_p (L)}{q_{all,net} + h(\gamma_s - \gamma_c)}$$

Equation 6.11

Where:

- b foundation width [m]
- P_t Roof load per unit length [kN/m]
- W_p product load on tank bottom [kPa]
- L distance from the tank shell to inside edge of the ringwall [m]
- $q_{all,net}$ net allowable soil bearing under ringwall [kPa]
- h height of the ringwall [m], 1.5m
- γ_s unit weight of the soil [kN/m³]
- γ_c concrete unit weight [kN/m³], 24 kN/m³

Net bearing pressure $q_{all,net}$ is referred to the bearing capacity calculated without considering the overburden pressure acting at the sides of the foundation. The equation basically represents an equilibrium between the resisting forces and the acting forces.

- Resisting forces: net bearing capacity at the bottom of the ringwall minus the effective weight of the concrete section
- Acting forces: the weight of the tank's roof plus a percentage of the product weight

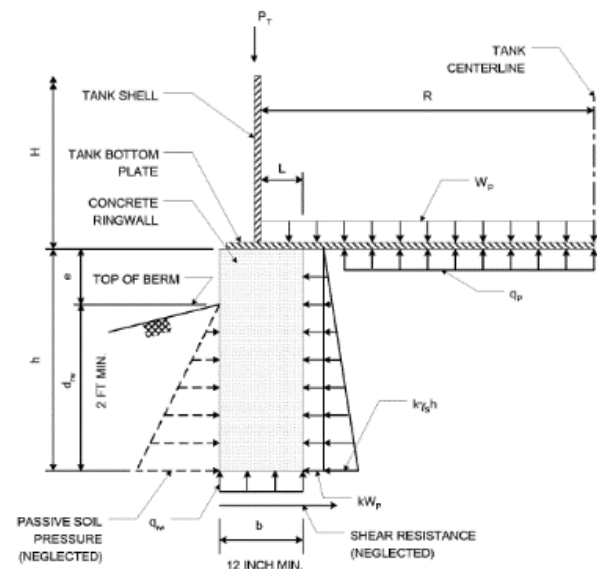


Figure 16 "Ringwall loading scheme (PIP STE 03020)"

The L dimension, gives the amount of the product pressure transferred from the tank to the ringwall; this dimension should be reduced as much as possible to reduce the acting forces pressure due to the tank content, however this length has to be wide enough to avoid structural failure mechanism of spalling which could occur at the edge of the ringwall. In according to PIP STE 03020 values of L , based on experience, are defined. For tanks, with a diameter lower than 24 meters, it is suggested to use a minimum value of $L = 10 \text{ cm}$.

From my point of view Equation 6.11 is not really reliable because it does not give limitation to the value of L , it can be argued that if this length is bigger than 1 meter, then the contribution of the product weight acting on the foundation will be bigger than the liquid weight W_p .

In this case studied, net bearing capacity $q_{all,net}$ is referred to the undrained case. The undrained bearing capacity without considering the stabilizing surcharge at the sides of the foundation has the following formulation:

$$q_{all,net} = (\pi + 2)c_u b_c s_c i_c \tag{Equation 6.12}$$

- c_u undrained cohesion
- b_c inclination of the foundation base = 1
- s_c shape factor = 1.2 for circular and square footing
- i_c inclination factor

$$i_c = \frac{1}{2} \left(1 + \sqrt{1 - \frac{H}{A'c_u}} \right) \tag{Equation 6.13}$$

6.3.1 Case Studied

Considering an undrained cohesion of 40 kPa , and a height of the foundation of 1.5 meters. It is possible to calculate the foundation width of the ringwall. The foundation height of 1.5 m , has been choice in according to the company. This is done to have a comparison between this work and the real design. Minimum foundation width is dimensioned to allow bolt anchorages between the tank and the ringwall. From paragraph 4.7.3, Equation 4.5, the minimum outside length of the tank should be 450 mm this length should be increased of 100 mm , to considered the distance between the outside of the shell and the inside of the ringwall, plus the shell thickness 9 mm . The minimum ringwall width has to be 600 mm . From Equation 6.11, considering all the load combinations in according to API 650 and Eurocode it is possible to calculate the minimum width of the ringwall by considering the

Table 30"Ringwall width section, according to Eurocode Load Combinations, case studied"

		Eurocode Ringwall Width							
		APPROACH I		APPROACH II	APPROACH III	APPROACH I		APPROACH II	APPROACH III
		Comb 1	Comb 2			Comb 1	Comb 2		
D	Foundation Width Used [m] min	0.04	0.06	0.09	0.06	0.04	0.06	0.08	0.06
	Foundation Height [m]	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
I	Foundation Width Used [m] min	0.04	0.06	0.09	0.06	0.04	0.06	0.08	0.06
	Foundation Height [m]	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
S	Foundation Width [m]	0.04	0.06	0.08	0.06	0.04	0.06	0.08	0.06
	Foundation Height [m]	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
W_F	Foundation Width Used [m] min	0.04	0.06	0.09	0.06	0.04	0.06	0.08	0.06
	Foundation Height [m]	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
W_E	Foundation Width Used [m] min	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
	Foundation Height [m]	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
S_F	Foundation Width [m]	0.06	0.10	0.13	0.11				
	Foundation Height [m]	1.50	1.50	1.50	1.50				
S_E	Foundation Width [m]	0.01	0.01	0.01	0.01				
	Foundation Height [m]	1.50	1.50	1.50	1.50				
Minimum Design Foundation Width [m]									0.15

equilibrium between the vertical loads and the bearing capacity. The results are summarised in Table 30 and Table 31 respectively.

In according to the API 650 the foundation size has been calculated using unfactored loads and reduced the bearing capacity of the soil, in according to each load case. Accordingly to Table 30 the nomenclature used for the load combination is:

- *D* Liquid Discharge (Self weight, Liquid filling, Wind)
- *I* Imposed load (Self Weight, Liquid filling, Imposed load, Wind)
- *S* Snow load (Self Weight, Liquid filling, Snow)
- *W_F* Wind and Full (Self Weight, Liquid filling full tank, Wind, Imposed load)
- *W_E* Wind and Empty (Self Weight, Liquid empty tank, Wind, Imposed load)
- *S_F* Seismic Full tank (Self Weight, Seismic Action, Liquid filling full tank, Imposed load)
- *S_E* Seismic Empty tank (Self Weight, Seismic Action, Liquid empty tank, Imposed load)

Table 31 "Ringwall width section according to API 650 Load Combinations, case studied"

Accordingly to Table 31 the nomenclature used for the load combination is:

- 1 Fluid and internal pressure (Self weight, Liquid Filling)
- 3 Wind and Internal pressure (Self weight, Wind)
- 5a) Gravity Load (Self Weight, Imposed load)
- 5b) Gravity Load (Shell Load, 0.4 Roof Load)
- *S* Seismic Full tank (Self Weight, Seismic Action, Liquid filling full tank, Imposed load)

API 650 Ringwall Width		
1	Foundation Width Used [m] min	0.11
	Foundation Height [m]	1.50
3	Foundation Width Used [m] min	0.03
	Foundation Height [m]	1.50
5 a)	Foundation Width [m]	0.11
	Foundation Height [m]	1.50
5 b)	Foundation Width Used [m] min	0.08
	Foundation Height [m]	1.50
S	Foundation Width [m]	0.11
	Foundation Height [m]	1.50
Minimum Design Foundation Width [m]		0.15

For both the norms, full filled tank with earthquake load controls the foundation width.

Difference between sections in according to the wind load combination is that the Eurocode considers the characteristic value of the wind load to be multiplied by a factor equal to 0.6, then the reduction of almost 40% between the two norms is plausible.

6.4 Circular Footing Stress

Assessment

In according to EN 1997-1:2004, settlements should be calculated until a depth at which the effective vertical stress due to the foundation load is 20% of the effective overburden stress, for Figure 17 this depth corresponds to $I_z = \sigma_z/q = 0.2$.

The influence factor to assess the stress changing at the central point of the foundation can be analytically calculated by using the following equation:

$$I_s = \left[1 - \left(\frac{1}{1 + \left(\frac{a}{z}\right)^2} \right)^{\frac{3}{2}} \right]$$

Equation 6.14

- a foundation radius
- z depth of the calculation point

In according to the solution provided by Boussinesq, Figure 17, to compute the induced stresses due to an applied external load the depth of interest is until a ratio $z/R_0 = 2.5$. Since the foundation width of the ringwall is 0.6 meters and since the tank radius is 4 meters, the depth of interested for the settlement calculation will be 11.5 meters below the tank bottom, since the ringwall 1.5 meters height the soil layer interested extend until a depth of 13 meters, from the ground surface. To work out the stresses the soil mass is divided in layers, generally they are thinner in proximity of the ground level and thicker while the depth increases.

Stresses in correspondence of the centre of the tank's foundation are graphically represented in Graph 21 and Graph 21.

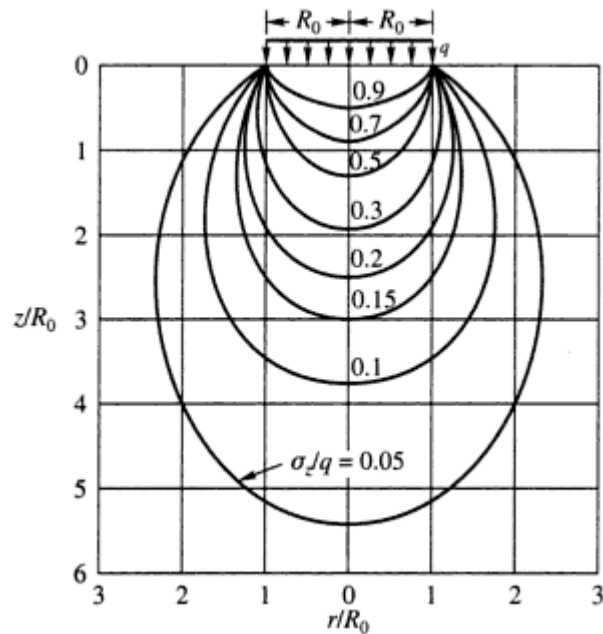
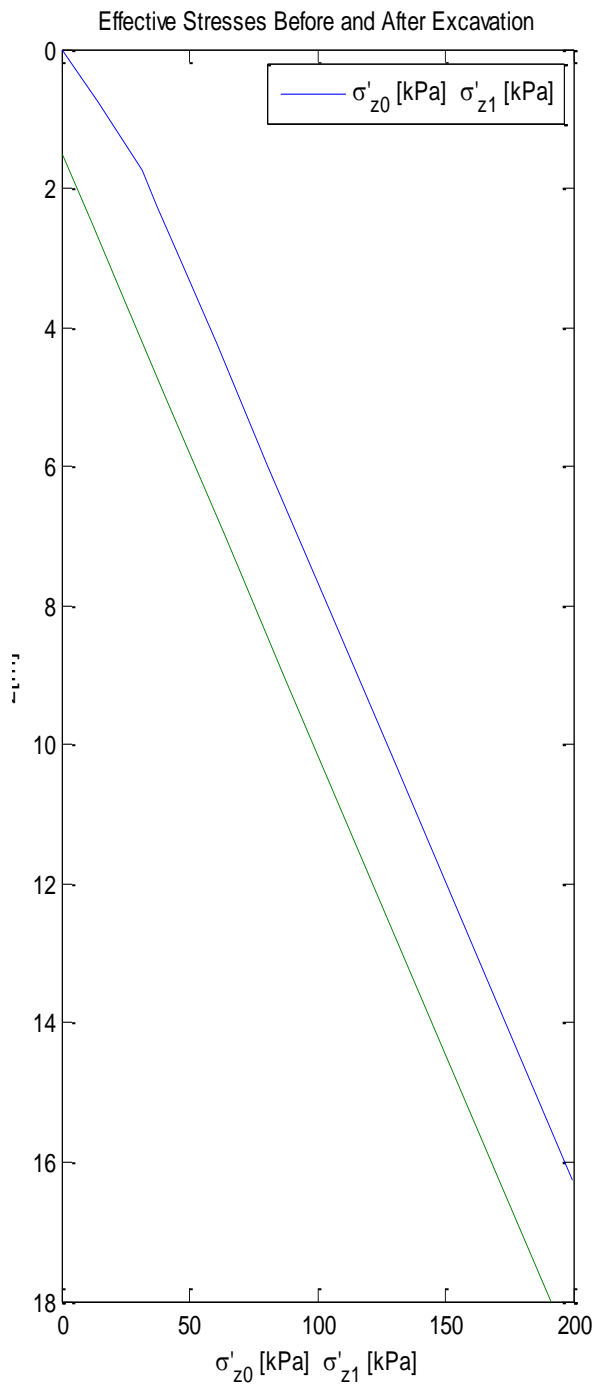
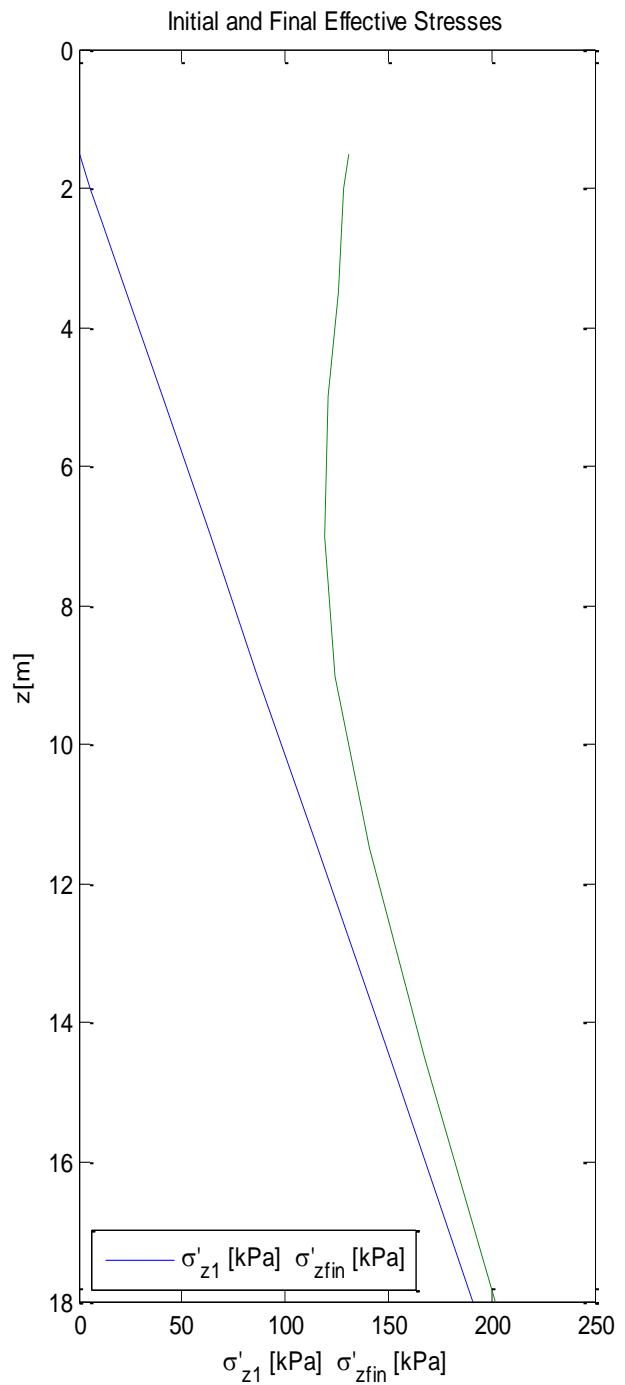


Figure 17 "Pressure bulbs for vertical uniform load, circular footing (Coduto, Yeung, & Kitch, 2011)"



Graph 21 "Effective stresses at the centre of the foundation tank, before and after the excavation, case studied"



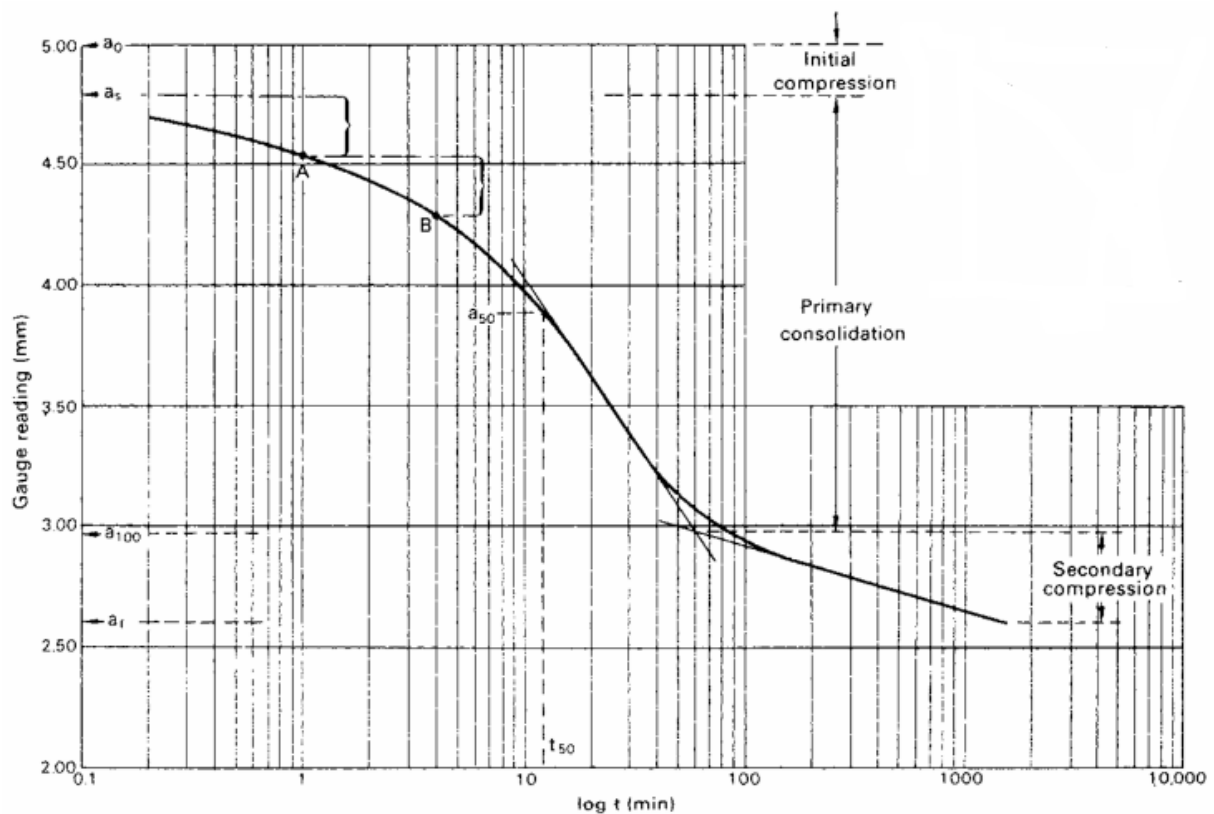
Graph 21 "Effective stresses at the centre of the foundation tank, before and after the tank's positioning, case studied"

6.5 Settlements

Magnitude of settlements is related to the type of soil, generally higher settlements occurs when the soil is normally consolidate rather than when is over consolidate. Considering deformation within a saturated soil layer with low permeability, the process can be schematized in the following manner, in according to Graph 22 (Craig, 2004):

- During loading, overpressure in the water within soil's pores increases, but since the low permeability of the soil mass, the water cannot escape from the pores and this situation is called undrained conditions. Settlement is called immediate settlement.
- Due to the overpressure in the water within the pores of the soil mass, the water pressure starts to be dissipated by flowing out from the pores, the overpressures from the water are transferred to the soil grains and the effective pressure increases, at the same time there are settlements within the soil layer called consolidation settlements.
- Once the overpressures have been dissipated settlements continue to occur, this process is called creep and they are settlements occurring during a constant load (Secondary Settlements).

Settlements are generally assessed by using the one dimensional Terzaghi's theory, once the increment of the stresses at the centre of the soil layer are determined, it is possible to calculate the settlements. The method uses parameters generally worked out by using the oedometer test, the values worked out are under the hypothesis that the deformations occur just vertically.



Graph 22 "One dimensional consolidation curve showing three independent physical processes- instantaneous strain (initial), primary consolidation and secondary compression (Craig, 2004)"

The calculation of the consolidation settlements for a normally consolidate cohesive material can use the following equation:

$$S_c = H_0 \frac{C_c}{1 + e_0} \log \frac{\sigma'_{v0} + \Delta\sigma_z}{\sigma'_{v0}} \quad \text{Equation 6.15}$$

- C_c compression index
- σ'_{v0} initial effective stress
- $\Delta\sigma_z$ stress increment
- e_0 initial void ratio value, relative to σ'_{v0}
- H_0 layer thickness

If the soil layer is over consolidate, for values of the increments of pressure which are lower than the consolidate pressure the recompression index c_r can be used rather than C_c .

$$S_c = H_0 \frac{c_r}{1 + e_0} \log \frac{\sigma'_{v0} + \Delta\sigma_z}{\sigma'_{v0}} \quad \text{Equation 6.16}$$

For secondary settlements it is possible to use the following formula:

$$S_s = \frac{C_\alpha}{1 + e_0} H \log \left(\frac{t}{t_{100}} \right) \quad \text{Equation 6.17}$$

- C_α secondary compression index
- t_{100} is the time required to have 100% of consolidation
- t is the time considered after consolidation is ended

In according to this model secondary settlements are calculated just after consolidation is reached.

6.6 Steel Reinforcements

Steel reinforcements have been designed just for hoop tension and twisting moment acting on the foundation. These tensional mechanisms are stated in API 650, while they are not listed in the Eurocode.

Hoop tension is the results of the lateral earth pressure developed in the inside ringwall face while twist moment acts around the longitudinal axis of the ringwall and it is generated by the eccentricity between the vertical load and the centroid of the mass of the foundation.

In according to the Eurocode, bearing capacity and section dimensioning of the foundation, can be checked by using the same load combination cases. Instead the foundation dimensioning accordingly to the American norms, has to be performed by using the ACI 318, this is the reference norm for concrete structures. This norm uses the allowable strength design method which is different from the allowable stress design method used by the API 650, due to this incongruity, load combinations used in API 650 are not suitable for the foundation dimensioning. Accordingly to PIP STC01025, load factors can be used to correlate the combination loads from the allowable stress design method to allowable strength design method.

In according to (ASCE 7-02:2002,p.239):

“Some of these specifications are based on allowable stress design, while others employ strength design. In the case of allowable stress design, design specifications define allowable stresses that may not be exceeded by load effects due to unfactored loads, that is, allowable stresses containing a factor of safety. In strength design, design specifications provide load factors and, in some instances, resistance factors.”

6.6.1 ACI 318 Load combinations

Load combinations are summarized in the table below Table 32. These combinations loads have been taken from PIP STC 01015.

Table 32 “Combination Load Cases, ACI 318 (PIP STC 01015)”

Loading Combinations – ACI 318		
Load Comb	Load Combination	Description
1	$D_s + D_0$	Operating Weight
2	$D_s + Dt + Pt$	Test Weight + Test Pressure
3	$D_s + (De \text{ or } D_0) + W$	Empty or Operating Weight + Wind
4	$D_s + (De \text{ or } D_0) + W$	Empty or Operating Weight + Wind
5	$D_s + D_0 + (L \text{ or } S)$	Operating Weight + Live
6	$D_s + (De \text{ or } D_0) + 0.4(L \text{ or } S)$	Empty or Operating Weight + Live
7	$D_s + D_0 + 0.1 S + E_0 + 0.4P_i$	Operating Weight + Snow + Earthquake + Internal Pressure
8	$D_s + D_0 + 0.1 S + E_0$	Operating Weight + Snow + Earthquake

- D_e Empty Weight of the Tank
- D_0 Structural Weight of the tank plus the maximum weight of the content during normal operations

- D_s Structure Dead Load is the weight of materials forming the structure (not the empty weight of process equipment, vessel, tanks, piping, nor cable trays), foundation, soil above the foundation resisting uplift, and all permanently attached appurtenances.
- D_t Test Weight Load, it is generally the weight of test medium used (water)
- E_0 Earthquake Load
- L Live Load
- P_t Test pressure load (Generally for structural design)
- S Snow Load
- W Wind Loas

Snow Load has not been considered in this design, the reason of this choice is because the Iraqi area is not considered as snowy (UFC, 2013).

In according to PIP STE 03020, there are safety factors which have to be used in load combinations, shown in Table 32, to tranform the loads worked out by using the ASD to the allowable strength desing method. In particular the product and the self weight of the structure have to be multiplied by a factor equal to 1.2, while the seismic load by using a factor of 1.4. The vertical seismic load should be calculated by multiplying the vertical load by the vertical acceleration coefficient and further multiplying the resulting value by the factor of safety.

Load combinations have been attached in Appendix G.

6.6.2 Twist moment

Eccentric loads from the tank shell and product load, create twisting moment on the ringwall foundation. The structural scheme of the equilibrium around the torsional axis can be schematized in according to Figure 18. The twist moment M_T for dead and fluid load can be so calculated:

$$M_T = P_t x_1 + W_p L x_2 \quad \text{Equation 6.18}$$

- M_T applied uniform twist moment on the ringwall, per unit length [kNm/m]
- P_t tank shell and roof load per unit length [kN/m]
- W_p pressure of product load on tank bottom [kPa]
- L distance from the tank shell to inside edge of the ringwall [m]
- x_1 x_2 eccentricity of the loads

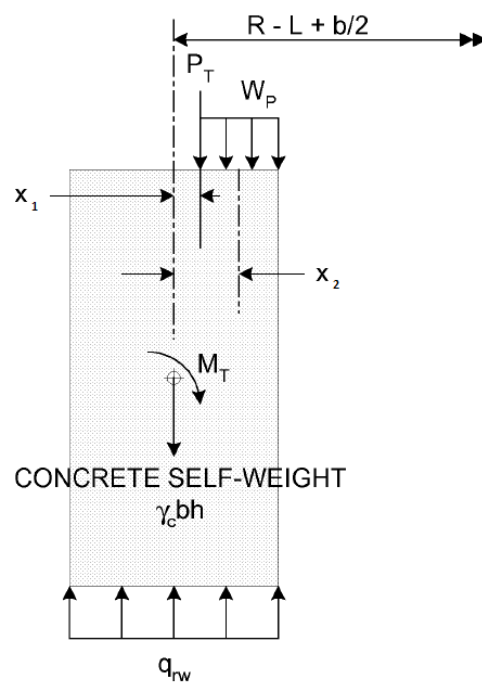


Figure 18 "Equilibrium scheme twisting moment (PIP 03020:2005)"

Regarding seismic and wind load, the twist moment is not uniformly distributed around the ringwall foundation, the scheme of the load distribution is shown in Figure 19.

6.6.2.1 Seismic Twist moment

Regarding the seismic load in according to PIP 03020, the vertical load due to the shell and the stored fluid can be calculated by multiplying the vertical load by one plus 0.4 the vertical seismic acceleration coefficient and then the twisting moment is calculated from the equivalent vertical seismic force.

The total vertical load $W_{Tot,seismic}$ generated by the seismic load is the sum of the shell load, roof load and product load multiplied by $1 + 0.4 A_v$ (vertical acceleration coefficient A_v).

$$W_{Tot,seismic} = (W_p + W_{shell} + W_{roof})(1 + 0.4 A_v)$$

Equation 6.19

6.6.2.2 Wind Twisting Moment

With regards to the wind twisting moment, it can be split in two vertical components: one generated by the product load and the second from the shell.

Considering the superposition of the effects it is possible to sum up these two components.

The superposition of the effects can be used assuming if the system considered has a static distribution of the forces and the section behaves elastically.

Regarding the twisting moment generated by the liquid content can be used the static scheme shown in

Figure 18, while for the wind moment it can be calculated the resultant vertical force generated at the edge of the shell.

The vertical component due to the wind moment can be calculate by dividing the moment for the tank diameter and dividing it by the circumference of the ring wall, (i.e. force P_T Figure 18). The twisting moment can be calculated in according to Figure 18.

6.6.2.2.1 Longitudinal Moment due to the Twisting moment

Once the torsional moment M_T is calculated, it is possible to transform it into longitudinal M , in particular it can be used the following equation:

$$M = M_T \left(R - L + \frac{b}{2} \right) \tag{Equation 6.20}$$

- M_T applied uniform twist moment on the ringwall, per unit length [kNm/m]
- R, L, b in according to Figure 18

Equation 6.20 is taken from (PIP STE03020, p.18), in reference to *Roark's Formulas for Stress and Strain (1989)*.

Stirrups are also suggested to be used to resist at the torsional twist moment and the provide support for the bottom and top steel bars. However stirrups are not designed in this thesis (PIP STE03020, p.18).

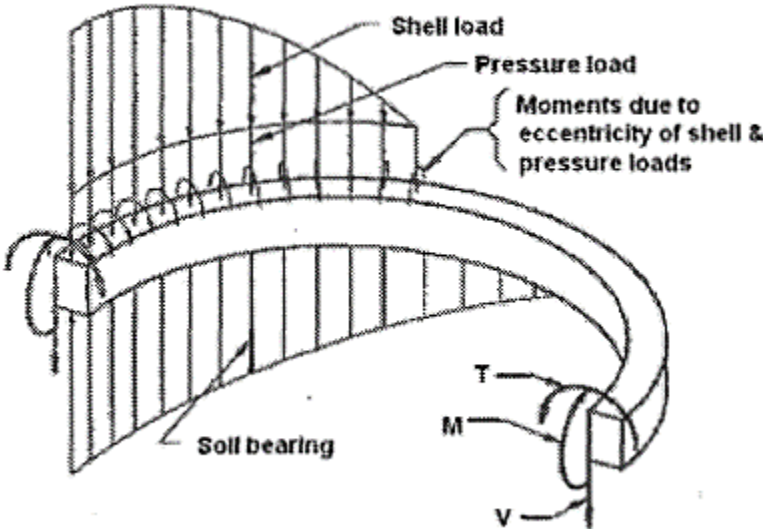


Figure 19 "Load distribution under seismic or wind load (PIP 03020:2005)"

6.6.2.3 ACI 318 Reinforcement Required

The longitudinal steel area, used in this case can be calculated from:

$$A_{st_{wist}} = \rho bd \quad \text{Equation 6.21}$$

- ρ ratio of A_s to bd
- b width of the foundation [m]
- d foundation height [m]

The ratio between the steel to the concrete is defined as:

$$\rho = \frac{0.85f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right) \quad \text{Equation 6.22}$$

- f'_c compressive strength of concrete [MPa]
- f_y steel yield strength
- R_n concrete beam design ratio

$$R_n = \frac{M_u}{\phi bd^2} \quad \text{Equation 6.23}$$

- ϕ resistance factor 0.9
- M_u maximum moment from factored load

6.6.2.4 Eurocode Reinforcement Required

In according to the Eurocode EN 1992-1-1, the steel reinforcement area for longitudinal moment can be calculated as follows:

$$A_{st_{wist}} = \frac{M}{0.9f_{yk}z} \quad \text{Equation 6.24}$$

- M acting moment
- f_{yk} characteristic yield steel strength
- z distance between the resultant compressive and tensile force

$$z = \frac{d[1 + \sqrt{1 - 3.529K}]}{2} \quad \text{Equation 6.25}$$

- d distance between the lower steel area and the top of the beam section
- K normalized bending resistance

$$K = \frac{M}{f_{ck}bd^2} \quad \text{Equation 6.26}$$

- f_{ck} characteristic compressive cylinder strength of concrete after 28 days
- b base of the beam section

6.6.3 Hoop Tension

Axial tension T_h in the ringwall is generated by lateral earth pressure, this pressure is the result of the product surcharge and the backfill within the ringwall foundation. The model used is in according to PIP STE 03020.

$$T_h = Rhk \left(W_p + \frac{\gamma_s h}{2} \right) \quad \text{Equation 6.27}$$

- R tank radius [m]
- h height of the ringwall [m]
- k coefficient of lateral earth pressure
- W_p product load on tank bottom [kPa]
- γ_s soil unit weight [kN/m³]

Regarding the soil pressure developed along the inside face of the ringwall, it can be calculated by using the lateral earth coefficient at rest. For granular soil, it is possible to use the Jaky's formula.

$$K_0 = 1 - \sin \phi \quad \text{Equation 6.28}$$

Following assumptions are used:

- Vertical earth retaining wall
- Smooth wall in which the interface between the wall and the soil is frictionless
- The supported soil is homogeneous and isotropic
- The soil is loose and originally in an at-rest state
- Lateral earth pressure must be applied to effective stress
- Critical slip plane are oriented at $45 \pm \phi' / 2$

When a beam is subject to tensile force, it can be assumed that the concrete does not resist to traction. However the possibility of cracking should be investigated. For the case studied in this thesis, will be assumed that just steel reinforcements will resist to tensile stresses excluding the contribution of the concrete section.

6.6.3.1 ACI 318 Hoop Stress Reinforcement Required

The longitudinal steel area A_s can be calculated in accordingly the following equation.

$$A_s = \frac{1.6 T_h}{0.9 f_y} \quad \text{Equation 6.29}$$

- f_y yield strength of the steel reinforcement

Where 1.6 is the soil load factor while 0.9 is the tension coefficient factor.

6.6.3.2 Eurocode Hoop Stress Reinforcement Required

In according to the Eurocode EN 1992-1-1:2005, a concrete reinforced section, should satisfy the following equilibrium:

The design ultimate resistance of the net cross section

$$A_{net} = \frac{\gamma_{M2} N_{u,Rd}}{0.9 f_y} \quad \text{Equation 6.30}$$

- f_y Ultimate strength

Recommended partial material factor:

- $\gamma_{M2} = 1.25$

Maximum Steel/Concrete ratio 4%.

The steel used from both the codes is directly provided by each code respectively. Then there has been the attempt to use a steel which almost the same yield strength. In particular has been used the A36 steel in according to API650 with a yield strength of 248 MPa while a S235 steel in according to EC3 with a yield strength of 235MPa.

6.6.4 Steel Area API 650 and EN 1993 comparison

The steel area worked out are:

Table 34 "Steel Section Areas in according to Eurocode load combinations, case studied"

	APPROACH I		APPROACH II	APPROACH III	APPROACH I		APPROACH II	APPROACH III	
	Comb 1	Comb 2			Comb 1	Comb 2			
Load Cases	Hoop Steel Area [mm²]								
	D	3743.2	3842.9	3743.2	3897.1	3723.5	3827.7	3723.5	3876.6
	I	3743.2	3842.9	3743.2	3897.1	3723.5	3827.7	3723.5	3876.6
	S	3743.2	3842.9	3743.2	3897.1	3723.5	3827.7	3723.5	3876.6
	W_F	3743.2	3842.9	3743.2	3897.1	3723.5	3827.7	3723.5	3876.6
	W_E	766.5	743.8	766.5	798.0	746.7	730.6	746.7	777.4
	Seismic								
	S_F	3739.4	4000.4	3739.4	3893.2	Max Hoop Steel Area [mm²]			4000.4
	S_E	598.5	618.2	598.5	623.2				
	Load Cases	Twist Steel Area [mm²]							
D		12450.2	12199.2	12450.2	12450.2	12353.2	12127.8	12353.2	12353.2
I		12450.2	12199.2	12450.2	12450.2	12353.2	12127.8	12353.2	12353.2
S		12450.2	12199.2	12450.2	12450.2	12353.2	12127.8	12353.2	12353.2
W_F		12645.0	12367.0	12645.0	12645.0	12547.6	12295.3	12547.6	12547.6
W_E		923.3	701.9	923.3	923.3	845.8	644.8	845.8	845.8
Seismic									
S_F		13049.8	13676.8	13049.8	13049.8	Max Twist Steel Area [mm²]			13676.8
S_E		594.9	440.1	594.9	594.9				
Total Longitudinal Area Hoop and Twist Mechanism [mm²]								17677.2	
Steel/Concrete Ratio [mm²]								2.0%	

Table 34 "Steel Section Area in according to ACI 318 load combinations, case studied"

Hoop Steel							
Load Comb	1	2	3	4	5	6	8
T_h [kN]	541	541	541	541	547	544	567
Steel Area [mm ²]	3849	3849	3849	3849	3892	3866	4031
Twist Steel							
A_s,twist [mm ²]	11215	11215	11224	11224	11367	11276	13128
Total Longitudinal Steel Area [mm²]							17159
Steel/Concrete Area %							1.9%

To summarise the results, regarding the hoop tension, and the twisting moment steel sections, the steel section calculated in according to Eurocode is 3% bigger than the one calculated for API 650. However the yield strength of the steel in according to the Eurocode is 5.5% higher the one used by API 650.

6.6.5 Undrained Bearing Capacity

Regarding the bearing capacity, the undrained condition is verified in all the load combination cases, for both the standards.

Table 35 "Undrained bearing capacity, Eurocode, load combinations according to Equation 6.1, case studied"

		APPROACH I		APPROACH II	APPROACH III
		Comb 1	Comb 2		
D	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	112	100	112	104
	safety factor ratio	2.45	2.05	1.75	1.96
		CHECKED	CHECKED	CHECKED	CHECKED
I	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	112	100	112	104
	safety factor ratio	2.45	2.05	1.75	1.96
		CHECKED	CHECKED	CHECKED	CHECKED
S	Bearing Capacity Used [kPa]	277	206	198	206
	Bearing Pressure [kPa]	110	98	110	102
	safety factor ratio	2.52	2.11	1.80	2.02
		CHECKED	CHECKED	CHECKED	CHECKED
W_F	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	112	100	112	104
	safety factor ratio	2.45	2.05	1.75	1.96
		CHECKED	CHECKED	CHECKED	CHECKED
W_E	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	49	37	49	41
	safety factor ratio	5.60	5.57	4.00	4.95
		CHECKED	CHECKED	CHECKED	CHECKED

Table 36 " Undrained bearing capacity, Eurocode, load combinations according to Equation 6.2, case studied"

		APPROACH I		APPROACH II	APPROACH III
		Comb 1	Comb 2		
D	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	112	99	112	104
	safety factor ratio	2.46	2.06	1.76	1.97
		CHECKED	CHECKED	CHECKED	CHECKED
I	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	112	99	112	104
	safety factor ratio	2.46	2.06	1.76	1.97
		CHECKED	CHECKED	CHECKED	CHECKED
S	Bearing Capacity Used [kPa]	277	206	198	206
	Bearing Pressure [kPa]	109	97	109	102
	safety factor ratio	2.53	2.12	1.81	2.03
		CHECKED	CHECKED	CHECKED	CHECKED
W_F	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	112	99	112	104
	safety factor ratio	2.46	2.06	1.76	1.97
		CHECKED	CHECKED	CHECKED	CHECKED
W_E	Bearing Capacity Used [kPa]	274	204	196	204
	Bearing Pressure [kPa]	49	36	49	41
	safety factor ratio	5.64	5.61	4.03	5.00
		CHECKED	CHECKED	CHECKED	CHECKED

Table 37 "Undrained seismic stability analysis in according to Eurocode load combinations, case studied"

S_F	Bearing Capacity Used 1 [kPa]	204.90	138.30	146.36	147.28
	Bearing Pressure 1 [kPa]	133.69	122.69	133.69	132.70
	safety factor ratio 1	1.53	1.13	1.09	1.11
		CHECKED	CHECKED	CHECKED	CHECKED
check		-0.678	-0.823	-0.678	-0.775
		CHECKED	CHECKED	CHECKED	CHECKED
S_E	Bearing Capacity Used 1 [kPa]	274.52	204.12	196.09	204.00
	Bearing Pressure 1 [kPa]	54.86	44.43	54.86	48.79
	safety factor ratio 1	5.00	4.59	3.57	4.18
		CHECKED	CHECKED	CHECKED	CHECKED
check		-0.992	-0.990	-0.992	-0.993
		CHECKED	CHECKED	CHECKED	CHECKED

Undrained bearing analysis accordingly to API 650 should be verified in according to the following factor of safeties.

1. From 2.0 to 3.0 against ultimate bearing failure for normal operating conditions;
2. From 1.5 to 2.25 against ultimate bearing failure during hydrostatic testing;
3. From 1.5 to 2.25 against ultimate bearing failure for operating conditions plus the maximum effect of wind or seismic load.

Table 38 "Undrained bearing capacity according to API 650 load combinations, case studied"

1	Bearing Capacity Used [kPa]	276.80
	Bearing Pressure [kPa]	84.95
	safety factor ratio	3.26 CHECKED > 3
3	Bearing Capacity Used [kPa]	273.49
	Bearing Pressure [kPa]	37.59
	safety factor ratio	7.28 CHECKED > 2.25
5 a)	Bearing Capacity Used [kPa]	276.80
	Bearing Pressure [kPa]	35.01
	safety factor ratio	7.91 CHECKED > 3
5 b)	Bearing Capacity Used [kPa]	276.80
	Bearing Pressure [kPa]	34.76
	safety factor ratio	7.96 CHECKED > 3
S	Bearing Capacity Used [kPa]	258.10
	Bearing Pressure [kPa]	99.31
		2.60 CHECKED > 2.25

6.7 Drained Bearing Capacity

Drained bearing capacity is performed by using the Brinch-Hansen equation, the foundation dimension are taken from the dimensioning performed in according to the undrained analysis since it controls the failure mechanism.

Table 39 "Drained bearing capacity according to Eurocode load combinations Equation 6.1, case studied"

		APPROACH I		APPROACH II	APPROACH III
		Comb 1	Comb 2		
D	Bearing Capacity [kPa]	326	237	233	236
	Bearing Pressure [kPa]	69	67	69	69
	Safety Factor ratio	4.75	3.51	3.39	3.44
		CHECKED	CHECKED	CHECKED	CHECKED
I	Bearing Capacity [kPa]	326	237	233	236
	Bearing Pressure [kPa]	69	67	69	69
	Safety Factor ratio	4.75	3.51	3.39	3.44
		CHECKED	CHECKED	CHECKED	CHECKED
S	Bearing Capacity [kPa]	332	241	237	241
	Bearing Pressure [kPa]	67	66	67	67
	Safety Factor ratio	4.93	3.63	3.52	3.57
		CHECKED	CHECKED	CHECKED	CHECKED
W_F	Bearing Capacity [kPa]	326	237	233	236
	Bearing Pressure [kPa]	69	67	69	69
	Safety Factor ratio	4.75	3.51	3.39	3.44
		CHECKED	CHECKED	CHECKED	CHECKED
W_E	Bearing Capacity [kPa]	323	234	231	234
	Bearing Pressure [kPa]	5	3	5	5
	Safety Factor ratio	71.58	69.14	51.13	51.27
		CHECKED	CHECKED	CHECKED	CHECKED

Table 40 "Drained bearing capacity according to Eurocode load combinations Equation 6.2, case studied"

		APPROACH I		APPROACH II	APPROACH III
		Comb 1	Comb 2		
D	Bearing Capacity [kPa]	326	237	233	236
	Bearing Pressure [kPa]	68	67	68	68
	Safety Factor ratio	4.78	3.52	3.42	3.46
		CHECKED	CHECKED	CHECKED	CHECKED
I	Bearing Capacity [kPa]	326	237	233	236
	Bearing Pressure [kPa]	68	67	68	68
	Safety Factor ratio	4.78	3.52	3.42	3.46
		CHECKED	CHECKED	CHECKED	CHECKED
S	Bearing Capacity [kPa]	332	241	237	241
	Bearing Pressure [kPa]	67	66	67	67
	Safety Factor ratio	4.96	3.65	3.54	3.60
		CHECKED	CHECKED	CHECKED	CHECKED
W_F	Bearing Capacity [kPa]	326	237	233	236
	Bearing Pressure [kPa]	68	67	68	68
	Safety Factor ratio	4.78	3.52	3.42	3.46
		CHECKED	CHECKED	CHECKED	CHECKED
W_E	Bearing Capacity [kPa]	323	234	231	234
	Bearing Pressure [kPa]	4	3	4	4
	Safety Factor ratio	79.20	75.40	56.57	56.72
		CHECKED	CHECKED	CHECKED	CHECKED

Table 41 "Drained bearing capacity according to API 650 load combinations, case studied"

1	Bearing Capacity Used [kPa]	256.92
	Bearing Pressure [kPa]	84.71
	safety factor ratio	3.03 CHECKED > 3
3	Bearing Capacity Used [kPa]	243.43
	Bearing Pressure [kPa]	36.10
	safety factor ratio	6.74 CHECKED > 3
5 a)	Bearing Capacity Used [kPa]	244.57
	Bearing Pressure [kPa]	35.01
	safety factor ratio	6.99 CHECKED > 3
5 b)	Bearing Capacity Used [kPa]	244.57
	Bearing Pressure [kPa]	34.76
	safety factor ratio	7.04 CHECKED > 3
S	Bearing Capacity [kPa]	244.17
	Bearing Pressure [kPa]	88.73
	Safety Factor ratio	2.75 CHECKED>2.25

7. Evaluation & Results Discussion

The thesis had the intent to compare the Eurocode and the API 650 for the design of an Oil tank foundation. This evaluation chapter has the following structure: firstly there is a general view of how the norms are structured. Secondly the Allowable Stress Design and the Load Resistance Factor Design methods have been compared regarding their differences. In the thesis wind and seismic load analyses were performed. Differences between these methods have been noticed and discussed. The number of combination loads are also presented, because, in terms of number typology are different. Moreover three approaches of Limit State Design method considered have been evaluated. The possibility of the designer to picking one of them rather than the other, require discussing the results and making conclusions. The approaches are explained in paragraph 7.5.

In accordance to the case studied, the design of the following structural parts has been made: shell design, foundation section and longitudinal steel bars (for twisting and hoop stress only). Shell design will not be considered important in terms of comparison, because it was performed only to assess the vertical dead load acting on the foundation and only the API 650 method was used. It is important to underline during the reading of this chapter that a comparison was done in terms of the final design of the structural components. It is not suggested to compare loads and bearing capacity, worked out following the Eurocode and the API methods, because loads and strength material properties have been factored in different ways. Regarding the steel sections dimensioning they have been dimensioned in according to steel strength values provided directly by the norms considered, in particular ACI 318 and EN 1992, which are different.

7.1 European and American Codes

To make the design of a tank, European and American norms can be grouped in terms of: loads, steel design, concrete design and geotechnical design.

Regarding load determination, the Eurocode requires to use:

- EN 1990 to have a basis to the Eurocode (87 pages)
- EN 1991 describes the actions on the structures in particular:
 - EN 1991-4 load combination of Silos and Tanks (110 pages)
 - EN 1991-1-4 Action on Structures – General actions – Part 1-4: Wind actions (148 pages)

Regarding structural aspects design:

- EN 1992-1-1 Design of concrete structures Part1-1: General rules and rules for buildings (225 pages)
- EN 1993-1-6 Design of Steel structures-Part 1- 6: General – Strength and Stability of Shell Structures (98 pages)
- EN 1993-4-2 Design of Steel Structures Part 4- 2: Tanks (61 pages)

In according to geotechnical design:

- EN 1997-1 Geotechnical Design – Part 1:General Rules (171 pages)

For seismic analysis:

- EN 1998-1 Design of structures for earthquake resistance – part 1: General rules, seismic actions and rules for buildings (232 pages)

- EN 1998-4 Design of structures for earthquake resistance Part 4: Silos, Tanks and pipelines (83 pages)
- EN 1998-5 Design of structures for earthquake resistance Part 5: Foundations, retaining structures and geotechnical aspects (47 pages)

Regarding the design in according to the American standards the following norms have been used

- * API 650 Welded and Steel Tanks for Oil Storage (449 pages)
- * API 653 Tank Inspection, Repairs, Alteration and Reconstruction (114 pages)
- * ASCE 7 Chapter 26 & 27 Wind Load General Requirements (60 pages)
- * PIP SCE 01015 Structural Design Criteria (32 pages)
- * PIP STE 03020 Guidelines for Tank Foundation Design (83 pages)

Eurocode is very detailed, it covers a large range of types of structures, while API 650 is related just to tank design. In accordance to the case studied it has been seen that API 650, covers all the design parts except for wind load analysis (where, however, API 650 gives a high value of the wind load, which is stated can be applied in most cases). It may be suggested that API 650 makes the design work easier for the engineer which has to deal with less codes and the methods presented are more straight forward than the Eurocode ones.

7.2 Method used

Two different methods are encountered: Allowable Stress Design and the Load Resistance Design methods. Even if the ASD method is easier to be used because fewer factors are used, it is based on factor of safety. According to (Ullman, 2010), factor of safety method is not very precise and the tendency is to use it conservatively. Between the safety factor ranges that are suggested by the standard (e.g. API gives a range between 2 and 3 for bearing capacity assessment), the use of the highest or lowest value is not justified by the norm, the tendency of the designers is to be every time conservative, often overdesigning each components (Ullman, 2010). Even in (Ullman, 2010), an approach based on statistical measures, as the Eurocode, where material properties are based on reliability, gives to the designer a better feeling for just how conservative or not the design is.

7.3 Wind and Seismic Load

Such as analysed in chapter 5, Eurocode and API wind load methods are similar. In particular the way of how equations derive wind velocity pressure and the corresponding wind pressure are qualitatively similar (the equations make use of the same coefficients, except Eurocode which does not consider the hurricane coefficient). Some differences are related on how the resultant wind force is worked out, in fact, in the case studied, the resultant force determined by EN 1991-1-4 results 20 % lower than the one of API.

The Eurocode dedicates a code EN 1991-1-4:2004 to determine the wind load response, the method is quite straight forward and many factors are given to the particular case of a tank structure. API 650 requires to use as reference norm ASCE 7-10, which uses a method that is used for all structural typologies.

Regarding seismic load, EN 1998-4 and API 650 consider the same approach, the reaction system of the tank is split into two components, convective and the impulsive masses. The whole structural response is done by considering the overturning equilibrium of these masses multiplied by the distance of the centroids from the tank's base. The norms give values for these masses, in function of the tank's height diameter ratio. The elevation of the centroids of the impulsive and convective masses consider two cases, ringwall and the slab foundation case. Differences are found just when the centroid of the

impulsive mass referred to the ringwall is considered. In this case the difference is almost of 5 %, as shown in Graph 10.

The difference between the norms is related mostly when the acceleration coefficient is worked out for the convective and impulsive component, in particular two main differences can be noticed:

1. Regarding the convective and the impulsive response API spectrum consider higher damping.
2. API 650 considers the end of the period with constant spectral acceleration longer in time, $T_s > T_c$ (Graph 14). For Eurocode the dissipation of energy starts earlier in time.

The whole seismic response, which is a combination between the convective and impulsive component is worked out differently by the two norms. In particular API 650 considers the Square Root of the Sum of the Square (SRSS) of the component while Eurocode considers the algebraic sum. The SRSS summation is justified by the New Zealand Code which states that since the convective and the impulsive responses occur at two different times, SRSS can be a good approach to combine them together. In accordance with the SRSS theory this technique is used when it is required to calculate the effect of actions assuming that maximum intensity of them does not occur at the same time (Lee, 1980). This justifies the fact that even if the impulsive and the convective acceleration components of the Eurocode are lower than the API's ones (Table 25), the whole response is higher for the Eurocode rather than the API (Table 26).

Regarding seismic stability requirements for a mechanically anchored tank, API 650 requires, that the overturning check has to be fulfilled. Product and foundation behave as a rigid body and it over-turns about the toe and the factor of safety should be bigger than 2.

7.4 Load Combinations

In EN 1991-4:2003 and API 650 the load combinations are respectively seven and eight. The comparison between each load combination was done considering how the different loads are considered in each combination. Magnitude of the vertical loads worked out cannot be compared because each norm applies different factors on each of them.

Combination load not mentioned in API 650 but in the Eurocode:

- Seismic Load considering Empty Tank

Load combinations used in the API 650 and not mentioned in Eurocode are:

- Hydrostatic Test
- Wind load has two load combination
 - Wind load plus internal pressure
 - Wind load plus external pressure
- Gravity load for fixed Roofs with Suspended Floating Roof

According to the case studied, Eurocode does not consider the hydrostatic test load combination, however it has been performed for each load combination.

For API 650, internal and external pressure loads have not been considered, because for oil content it is not required that tanks are completely sealed, this assumption has been done considering the fact that many oil tanks are open on top (Long & Garner, 2003).

Considering the case studied, in terms of bearing capacity and to foundation sizing design, the most critical combination load for both norms is under seismic load with a fully filled tank. A difference between the Eurocode and the API, is that the Eurocode, besides considering load combination,

performs the analysis according to three different approaches, which have been also evaluated. The three approaches are explained in the next paragraph.

7.5 Eurocode approaches

Eurocode uses three approaches which differentiate each other in according to how partial factors are distributed among: loads, strength material properties and resistance mechanisms. They are summarized in Table 10.

Approach number one, DA1, is split into two “sub-approaches”, which have to be both fulfilled. The first, of the two, applies factors to the loads while the second reduces the material strength resistances.

Approach DA 2, increases the loads and reduces the resistance failure mechanism of the foundation.

Third approach, DA 3, applies factors to loads and material resistance, but different load factors are used if structural or geotechnical loads are considered.

According to the case studied, for all load cases, the second approach gives lower values of the safety factors.

In accordance to the case studied, the bearing capacity equilibrium, in according to all load cases, is compared in terms of undrained bearing capacity, since for the soil in the site considered, this is the most critical condition.

Undrained bearing capacity can be assessed by the following equation:

$$\text{Bearing Capacity} = (\pi + 2)c_u b_c s_c i_c + q \quad \text{Equation 7.1}$$

- b_c inclination foundation base factor if it is assumed equal 1 (foundation base is considered not inclined)
- c_u undrained cohesion
- i_c inclination load factor is function to the lateral and vertical load because function of the effective area then the eccentricity.

$$i_c = \frac{1}{2} \left(1 + \sqrt{1 - \frac{H}{A' c_u}} \right)$$

Equation 7.2

- H Horizontal Load
- A' Effective Area
- s_c is equal 1.2 for circular footing
- q lateral surcharge (load acting on the sides of the foundation)

The comparison between the approaches of the Eurocode, is performed by considering the ratio between bearing capacity and load applied (i.e. safety factor Equation 7.3) and by considering which of them gives the bigger foundation section.

Such as explained in Equation 2.17, factor of safety is so defined:

$$FS = \frac{\text{Bearing Capacity}}{\text{Load Applied}} \quad \text{Equation 7.3}$$

To analyse the factor of safety two terms have to be define: bearing capacity and load applied. Both of them change in according to the approach studied. To compare the approaches it can be suggested to look which combination gives the lowest safety factor.

7.5.1 Approach Comparison Foundation Sizing

Regarding the pre-dimensioning of the ringwall width, Equation 7.4 can be used. This equation is function of net bearing capacity, Equation 7.1 without the second term (q term).

$$b = \frac{P_t + W_p (L)}{q_{all,net} + h(\gamma_s - \gamma_c)} \quad \text{Equation 7.4}$$

Where:

- b foundation width [m]
- P_t Roof load per unit length [kN/m]
- W_p product load on tank bottom [kPa]
- L distance from the tank shell to inside edge of the ringwall [m]
- $q_{all,net}$ net allowable soil bearing under ringwall [kPa]
- h height of the ringwall [m], 1.5m
- γ_s unit weight of the soil [kN/m³]
- γ_c concrete unit weight [kN/m³], 24 kN/m³

Regarding the undrained bearing capacity, two cases in according to the Eurocode approaches should be checked. In particular DA3, where factors are applied to the undrained cohesion or DA2 where partial factor are applied to reduce the resistance of the failure mechanism (i.e. Bearing Capacity). The lowest bearing capacity is obtained when the factors are applied to reduce the resistance of the failure mechanism.

- Lower bearing capacity values occur when partial factors are applied to the resistance failure mechanism rather than when are applied to undrained bearing capacity. The inclination factor controls this mechanism: if we apply a safety factor to the undrained cohesion, the term inside the square root of Equation 7.2 does not reduce the inclination factor term i_c as much as when partial factor is applied to all Equation 7.2, in this last case gives a major reduction.

Moreover, for the case studied, the approaches which give the bigger foundation width are the one which apply higher factors to the later loads. This is because an increment of the lateral load gives a high reduction of the inclination factor.

- i_c is function of the effective area which is the function of the eccentricity, which is function of the magnitude of the lateral and vertical load. During this pre-dimensioning stage, the foundation is not defined yet, vertical load is low, in fact it represented by: tank shell, roof and content load (excluding a big component represented by the foundation weight). If lateral load is increased, (by a load combination factor), i_c decreases significantly.

Among all the approaches the one which gives the bigger section is the one that applies higher factors to lateral wind load, smaller values to the vertical load and resistance partial factors instead than partial factors to the strength properties of the soil, i.e. approach two.

7.5.2 Approach comparison General Failure mechanism

For the general failure mechanism, load combination approaches two and three give lower safety factors (Table 37) rather than approach one. However load combination two governs the failure mechanism. There are two reasons in according to this aspect:

- Firstly because in approach DA2, the weight of the soil within the ringwall (vertical load) is multiplied by 1.35 while approach DA3 considers geotechnical weight as unfactored, reducing then the bearing pressure
- Secondly, because in approach three factors of safety are applied to the undrained cohesion while in approach two they are applied to Equation 7.1, i.e. bearing capacity. According to Equation 7.1, when a partial factor is applied to the undrained bearing capacity, it reduces just the first term of the equation while when it is applied to the bearing capacity also the second term, (lateral surcharge term q) is reduced. The most conservative result is when factors are applied to both of the terms, i.e. DA2.

7.6 Steel Reinforcement

For steel reinforcements of foundation, two types of stresses have been considered. According to PIP 03020, steel reinforcements have been designed to resist hoop and twisting stresses. The reference American norm for steel-concrete structures, is the ACI 318, which uses a particular type of LRDF (Load Resistance Design Factor) method, called the strength design factor method, which applies factors to the loads and to the material strength parameters. For this reason it is not possible to use the same load combinations that were used for the API. It is important then to make new combination loads which can be taken from PIP STC 01015.

Regarding Eurocode the reference code used is EN 1992-1-1:2005, and the combination loads are the one used for the geotechnical design.

However in this analysis, it may be suggested, that a comparison between the steel section areas worked out by the yield strength of the steel taken from the Eurocode and ACI 318 cannot be done. The reason why is because steel material properties have to be taken from the norms which have factored their resistance, in fact, the yield strength properties of the two steel does not match. Structural steel material properties are documented in EN 10025 where for a steel S235 ($F_y = 235 \text{ MPa}$, $F_u = 360 \text{ MPa}$) while a similar steel for the ACI 318 is A36 ($F_y = 220 \text{ MPa}$, $F_u = 340 \text{ MPa}$).

8. Recommendation

In according to the case studied, the soil parameters gained are considered as preliminary soil investigation values. The difference between the soil parameters of the boreholes, shown in Appendix C require to perform more soil investigations.

Since in the site considered presence of rocks can be excluded, is suggested to use CPT test.

In particular is also required to perform oedometer tests, to characterise parameters as:

- Consolidation coefficient c_v
- Compression index C_c

Drained (CD) or consolidate undrained (CU) triaxial test at two different cell pressures to know effective friction angle and effective cohesion.

It is suggested to make an analysis of the foundation structure because the previous equations are just to assess a pre dimensioning.

Assessment of the preload embankment's stability should be performed as well.

Sliding assessment of the foundation should be performed.

Design aspects not considered, such as the stresses generated in the bottom of the shell plate due to the settlements (Wu & Liu, 2000) should be checked.

9. Conclusions

The first difference between Eurocode and API 650 is that the first is a code and the second a standard. Eurocode gives rules while API 650 gives a set of analysis methodologies more similar to a guideline.

Regarding the methods used, Eurocode is based on the Load Resistance Design method, which factorises load and resistance properties while API uses the Allowable Stress Design method based on the unfactored values. Since many properties have to be factorised, the amount of calculations in according to Eurocode is bigger.

Despite more calculations Eurocode it is considered more reliable than API because the factorised design load and resistance are based on reliability theory, while safety factor of ASD does not tell anything about probability of failure.

The norms have been compared on how they determine wind and seismic resultant loads. Regarding wind load, both use the same method, a difference was seen in the Eurocode method, which does not uses the hurricane gust factor.

Seismic load comparison has been done in according to two aspects: one referred to the reaction system considered and a second regarding the design spectrum of response.

Both norms consider the same reaction system, constituted by dividing the whole tank's mass into convective and impulsive mass. The methods used to define impulsive and convective components are the same for both norms. The difference between the norms is related to the response design spectrum.

- API spectrum considers an higher damping
- Eurocode spectrum considers the damping dissipation in the system to start earlier

It has been noticed that:

- If the time of the maximum impulsive response of the structure is high, then Eurocode, gives an higher impulsive acceleration response than the API
- If the time of the maximum convective response of the structure is high, API gives a higher convective response than the Eurocode

By analysing a particular case studied, the norms have been compared in terms of predimensioning the foundation size and steel reinforcement (just for hoop and twisting stress) of a tank structure. At the end of the predimensioning analysis, a big difference between the sections calculated in according to the norms was not seen.

In according to the case studied, a settlement analysis was also performed, however both the norms use the same method (unfactored load and unfactored resistance properties). Then the comparison has not been done.

Appendix A Soil Bearing Capacity Equation

Regarding soil bearing capacity equations, the Prandtl and Brinch Hansen equations are presented. They are the equations to assess the undrained and drained bearing capacity respectively.

The following symbols are used in this appendix

- A' design foundation effective area = $B'L'$
- b the design value of the factors for the inclination of the base
- B foundation width
- B' effective foundation width = $B - 2e$
- c' effective cohesion
- D embedment depth
- e eccentricity of the resultant action, with subscripts B and L
- i the inclination factors of the load,
- L foundation length
- L' effective foundation length = $L - 2e$
- m exponent in formulas for the inclination factor i
- N the bearing capacity factors
- q overburden or surcharge pressure at the level of the foundation base
- q' the design effective overburden pressure at the level of the foundation base
- s shape factor of the foundation base
- R vertical load
- α inclination of the foundation base to the horizontal
- γ' the design effective weight density of the soil below the foundation level θ direction angle of H
- ϕ' effective friction angle

A.1 Undrained Case

$$Q_f = 2P_p + 2C_u \sin \phi = 2P_p + BC \operatorname{tg} \phi \quad \text{Equation A.1}$$

$$\frac{R}{A'} = (\pi + 2)c_u b_c s_c i_c + q \quad \text{Equation A.2}$$

Where:

- b_c is the inclination factor of the foundation $b_c = 1 - \frac{2\alpha}{\pi+2}$
- s_c is the shape factor of the foundation
 - $s_c = 1 + 0.2 \left(\frac{B'}{L'}\right)$, for a rectangular shape
 - $s_c = 1.2$ for square or circular shape
- i_c the inclination factor of the load, caused by an horizontal load H;
 - $i_c = \frac{1}{2} \left(1 + \sqrt{1 - \frac{H}{A'c_u}}\right)$

With $H \leq A'c_u$

A.2 Drained Case

The drained bearing resistance of the foundation soil is so calculated:

$$\frac{R}{A'} = c'N_c b_c s_c i_c + q'N_q b_q s_q i_q + 0.5 \gamma' N_\gamma b_\gamma s_\gamma \quad \text{Equation A.3}$$

The bearing resistance factor:

- $N_q = e^{\pi \tan \theta'} \tan^2 \left(45 + \frac{\theta'}{2} \right)$
- $N_c = (N_q - 1) \cot \theta'$
- $N_\gamma = 2(N_q - 1) \tan \theta'$ where $\delta \geq \theta'/2$

The inclination factor of the foundation base:

- $b_c = b_q - (1 - b_q)/(N_c \tan \theta')$
- $b_q = b_\gamma = (1 - \alpha \tan \theta')^2$

The shape factor of the foundation

- $s_q = 1 + \left(\frac{B'}{L'} \right) \sin \theta'$ for a rectangular shape
- $s_q = 1 + \sin \theta'$ for a square or circular shape
- $s_\gamma = 1 - 0.3 \left(\frac{B'}{L'} \right)$ for a rectangular shape
- $s_\gamma = 0.7$ for a square or circular shape
- $s_c = (s_q N_q - 1)/(N_q - 1)$ for rectangular, square and circular shape

The inclination factor of the load, caused by a horizontal load H;

- $i_c = i_q - (1 - i_q)/(N_c \tan \theta')$
- $i_q = [1 - H/(V + A'c' \cot \theta')]^m$
- $i_\gamma = [1 - H/(V + A'c' \cot \theta')]^{m+1}$

$$m = m_b = \left[2 + \left(\frac{B'}{L'} \right) \right] / \left[1 + \left(\frac{B'}{L'} \right) \right] \text{ when H acts in the direction of } B';$$

$$m = m_b = \left[2 + \left(\frac{L'}{B'} \right) \right] / \left[1 + \left(\frac{L'}{B'} \right) \right] \text{ when H acts in the direction of } L';$$

In case where the horizontal load component acts in a direction forming an angle θ with the direction of L' ; m may be calculated by:

$$m = m_\theta = m_L \cos^2 \theta + m_B \sin^2 \theta$$

Appendix B Foundation Typology

In Table 42, advantages and disadvantages of ringwall and slab foundation are summarised, PIP 03020:2005.

Table 42 "Advantages and Disadvantages of Ringwall and Slab concrete foundation (PIP 03020)"

Foundation Type	Advantages	Disadvantages	Recommendations
Concrete Ringwall	<ol style="list-style-type: none"> 1. Provides level surface for shell construction 2. Minimized edge settlement 3. Easy levelling for tank grade 4. Minimized moisture under tank 5. Retains fill under tank and prevents loss due to erosion 6. Distributes concentrated shell load well 7. Provides greatest assurance of meeting elevation tolerances around tank circumference 8. Better able to transfer shell loads to the supporting soil 9. Minimizes edge settlements and consequently shell distortions – very important problems to avoid for trouble-free operation of tanks with floating roof 	<ol style="list-style-type: none"> 1. Can be expensive, depending on location 2. May not be suitable for tanks on poor soils. 3. Ringwall must be reinforced 	<p>Preferred foundation type for tanks larger than 20 feet (6.1m) in diameter. Can also be used for small-diameter tanks if anchorage is not required.</p> <p>Use on good soil or proper prepared intermediate soils.</p> <p>Concrete ringwall is the preferred foundation for the following:</p> <ol style="list-style-type: none"> a. All large tanks b. Tanks where the surface soil is noncohesive, such as loose sand c. Tanks where significant settlement is anticipated d. All floating roof tanks larger than 30 feet (9.1 m) in diameter to protect against differential settlement-caused problems with annular space and tank shell
Pile Foundations	<ol style="list-style-type: none"> 1. Minimizes total and differential settlement 2. No separate bottom pad required 3. Allows for tank anchorage against wind and earthquake overturning 4. Leak detection and containment can be incorporated 	<ol style="list-style-type: none"> 1. Most expensive foundation type 2. More complex design than other types 3. Good soils information essential 4. Cathodic protection more difficult to install 	<p>Use for all tank foundation on poor soils where no other foundation type is feasible</p>

Appendix C Soil Test data

Test results are included in this appendix, they are relative to three boreholes BH1, R9 and R10 respectively.

Table 44 "Soil Data Borehole 1 BH1 Basra site"

Sample		Depth (m)		Index Properties			Particle size distribution and hydrometer analysis				SPT N value	Sym. Unit Class	Description of Soil	Unit Weight g/cm3		Strength Parameters				Chemical Tests						
Type	from	to	M.C. %	L.L. %	P.L. %	Clay %	Silt %	Sand %	Gravel %				Wet	Dry	UC test KN/m2	Direct test	Undrained		SO3 %	T.S.S. %	ORG %	Gyp %	CaCO3 %	PH	CL %	
																C	φ	Cu	φ	τ						
D	0	0.5		48	24								2.09	1.8				100	0	78	0.69		1.48	45		
U	0.5	1				59	41																			
S.S.	1	2.5				50	49	1							15						5.82					
U	3	3.5	43			53	43	4																		
S.S.	3.5	4		45	21																					
S.S.	6	6.5		48	20																					
S.S.	8.5	9				44	48	8																		
S.S.	11	11.5		49	17																					
S.S.	14	14.5		49	17																					
S.S.	16.5	17		24	17																					
S.S.	18.5	19		35	17																					
S.S.	22	22.5				47	43	25																		
S.S.	24.5	25				14	85	24																		
S.S.	27	27.5				31	69	31																		
S.S.	29.5	30				35	65	31																		
S.S.	32	32.5				25	75	32.5																		
S.S.	34.5	35				47	53	49																		
S.S.	37.5	38				34	66	34																		
S.S.	39.5	40				39	31	31																		

Table 44 "Soil Data Borehole R9 Basra site"

Sample		Depth (m)		Index Properties			Particle size distribution and hydrometer analysis				SPT N value	Sym. Unit Class	Description of Soil	Unit Weight g/cm3		Strength Parameters				Chemical Tests						
Type	from	to	M.C. %	L.L. %	P.L. %	Clay %	Silt %	Sand %	Gravel %				Wet	Dry	UC test KN/m2	Direct test	Undrained		SO3 %	T.S.S. %	ORG %	Gyp %	CaCO3 %	PH	CL %	
																C	φ	Cu	φ	τ						
D	0	0.5				15		78	7				1.73	1.6												
S.S.	0.5	1				11		77	12																	
S.S.	2.5	3				53	37	9	1																	
S.S.	4.5	5				48	40	12	7																	
S.S.	6.5	7				35	14																			
S.S.	9	9.5				14																				
S.S.	11.5	12				42	41	16	1																	
S.S.	14.5	15				25	9																			
S.S.	20	20.5				41	45	14																		
S.S.	22	22.5				36	15																			
S.S.	23	23.5				46	36	11	7																	
S.S.	26	26.5				43	18																			
S.S.	29	29.5				45	55																			
S.S.	32	32.5				44	54	2																		
S.S.	35	35.5				29	71																			
S.S.	37	37.5				32	68																			
S.S.	39.5	40				31	69																			

Appendix D Wind load calculation

The following calculations are done in according to EN 1991-1-4:2004 and to ASCE 7-10.

D.1 Wind Load EC1

Wind load action is determined from the basic values of wind velocity or velocity pressure which is the value relative to the mean characteristic 10 minutes wind velocity. In accordance to EN 1990 4.1.2 (7) P, these basic values are characteristic values having an annual probabilities of exceedence of 2% which is equivalent to a return period of 50 years. The response of the structure is calculated considering the peak velocity pressure q_p and the structural factors c_s and c_d .

The basic wind velocity is calculated as:

$$v_b = c_{dir}c_{season}v_{b,0} \quad \text{Equation D.1}$$

Where:

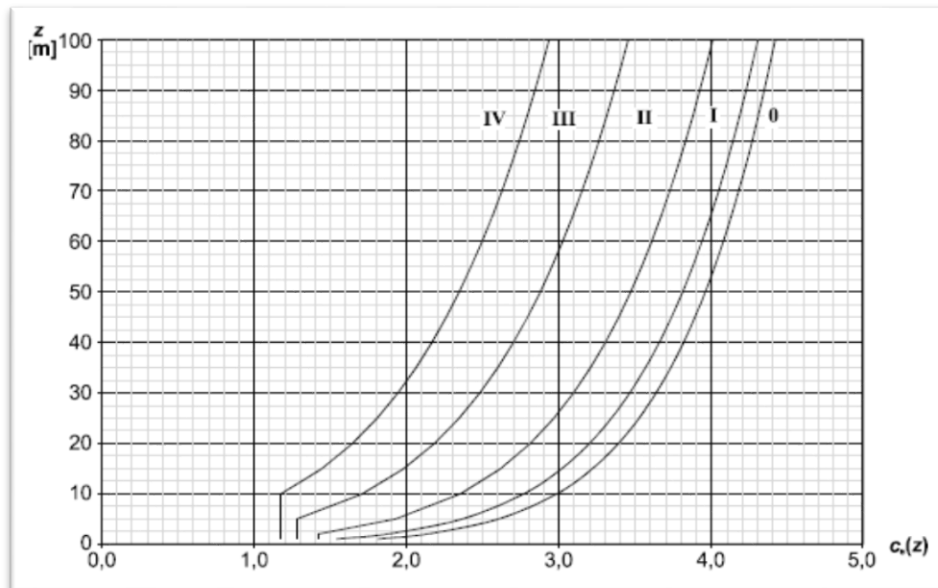
- $v_{b,0}$ calculated from a correlation with the three second gust velocity defined for the ASCE 7-10, $v_{b,0} = 35.2 \text{ m/s}$
- c_{dir} directional factor, 1.
- c_{season} season factor, 1.0

From the basic wind velocity it is possible to calculate the peak velocity pressure q_p at height z for the design the z height is equal to the height of the structure considered.

$$q_p(z) = c_e(z)q_b \quad \text{Equation D.2}$$

Where:

- $c_e(z)$ is the exposure factor for flat terrain the value can be taken from the following chart:



Graph 23 "Wind exposure factor $c_e(z)$ (EN 1991-1-4:2004)"

- q_b is the basic velocity pressure $= \frac{1}{2}\rho v_b^2$
 - ρ it is the air density, the recommended value is 1.25 kg/m^3

$$q_b = 774.64 \text{ kg/m}^3$$

The pressure distribution along the vertical axis of the tank, is calculated respect to two sections: the lower part, from the ground level until a height equal to the diameter ($q_{p,bottom}$) and the remain part ($q_{p,upper}$).

$$q_{p,bottom} = 1429.31 \frac{\text{Kg}}{\text{mS}^2} \qquad q_{p,upper} = 1592.1 \frac{\text{Kg}}{\text{mS}^2}$$

D.1.1 Wind pressure

The wind pressure on the tank's shell is split into external w_e and internal pressure w_i :

$$w_e = q_p(z_e)c_{pe} \qquad \text{Equation D.3}$$

Where

- c_{pe} is the external pressure coefficient

$$w_i = q_p(z_i)c_{pi} \qquad \text{Equation D.4}$$

Where

- c_{pi} is the internal pressure coefficient

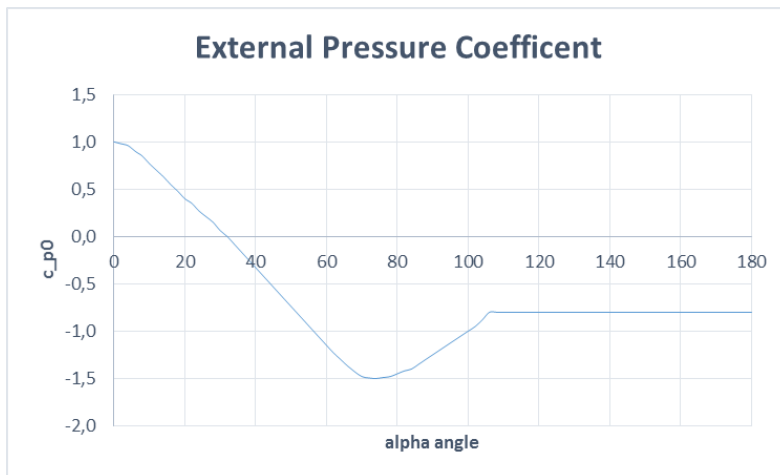
This coefficient is used to work out the variation of the external pressure around the shell circumference.

$$c_{pe} = c_{p,0}\Psi_{\lambda\alpha} \qquad \text{Equation D.5}$$

Where:

- $c_{p,0}$ is the external pressure coefficient without free-end flow, function of the Reynolds number and it changes along the circumferential profile in according to the rotational angle.
 - $Re = \frac{bv(z)}{v}$ Reynolds is calculated for the lower and the upper part of the shell. Then until an 8 meters of height and the upper, represented by remaining part of the shell. $Re_{bottom} = 2.55 \cdot 10^7$ and $Re_{top} = 2.69 \cdot 10^7$
 - $v(z)$ is the peak wind velocity changing with z . It is equal to $\sqrt{\frac{2q_p}{\rho}}$, the velocity will be calculated for the bottom and the upper part of the section. $v(z)_{bottom} = 47.8 \text{ m/s}$ and $v(z)_{top} = 50.5 \text{ m/s}$
- $\Psi_{\lambda\alpha}$ is the end-effect factor. It is 0.62 for the structure considered.

The variation of c_{p0} along the circumference is plotted in Graph 24.



Graph 24 "Wind coefficient of External Pressure (EN 1991-1-4:2004)"

D.1.2 Wind Force

The wind force acting on the structure is:

$$F_w = c_s c_d c_f q_p(z) A_{ref} \quad \text{Equation D.6}$$

Where:

- The coefficient c_s c_f can be considered equal 1 for structures lower than 15 meters.
- c_f is the force coefficient it is function of the roughness of the shell surface, in the present project it has been worked out the value for a cast iron and a rust surface.

$$c_{f,0} = 1.2 + \frac{0.18 \log\left(10 \frac{k}{b}\right)}{1 + 0.4 \log\left(\frac{Re}{10^6}\right)} \quad \text{Equation D.7}$$

- k is the equivalent roughness factor.

$$c_f = c_{f,0} \Psi_\lambda \quad \text{Equation D.8}$$

- A_{ref} is the reference area which is the section area perpendicular to the wind direction $80m^2$.

The total forces calculated for the lower and upper part of the section and considering two cases, in according to the roughness of the shell surface.

Table 46 "Resultant Wind Forces Eurocode (EN 1991-1-4:2004), case studied"

Resultant Wind Force		
	Cast iron shell	Rust shell
Bottom (1)	73015	80397
Top (2)	20347	22391
Tot	93362 [N]	102788 [N]

The design will be performed considering a rust surface and the total lateral force is 102788 N.

D.2 Wind Load ASCE 7-10

Such as for the determination of the wind load in according to EC1, the procedure of ASCE 7-10 considers to find the wind pressure firstly and then the resultant wind force.

D.2.1 Wind Pressure

Wind velocity pressure q_z is:

$$q_z = 0.613 K_z K_{zt} K_d V^2 I \quad \text{Equation D.9}$$

Where:

- K_z velocity pressure exposure coefficient 1.04
- K_{zt} topographic factor 1
- K_d wind directional factor 0.95
- V basic wind speed [m/s]
- I importance factor 1.15

D.2.2 Wind Force

For building with a height smaller than 18 meters, the horizontal force and the vertical uplift forces can be calculated. The horizontal force is defined as:

$$F_h = q_h (GC_r) A_f \quad \text{Equation D.10}$$

- q_h is the velocity pressure, calculated at height h
- A_f vertical projected area of the rooftop structure
- (GC_r) for rooftop structures and equipment, it can be reduced linearly from the value 1.9 to 1.0 as the value of A_f is increased from $(0.1Bh)$ to (Bh)

The vertical uplift force is:

$$F_v = q_h (GC_r) A_r \quad \text{Equation D.11}$$

- q_h is the velocity pressure of the equation above, calculated at height h
- A_r horizontal project are of the rooftop structure or equipment

(GC_r) 1.5 for A_r less than $(0.1BL)$ shall be permitted to be reduced linearly from 1.5 to 1.0 as the value of A_r is increased from $0.1BL$ to BL , where B is the horizontal dimension of the building and L is the horizontal dimension of the building

Table 47 "Resultant Wind Forces API 650, case studied"

Horizontal wind Force	123,349.59 N
Vertical wind Force	77,463.54 N

Appendix E Seismic map hazard (Eurocode)

This map represents the Middle East seismic hazard. The values reported express the peak ground acceleration, with a return period of 50-years and a probability of 10%. It has been provided by the U.S. Geological Survey National Earthquake Centre.

Seismic Hazard

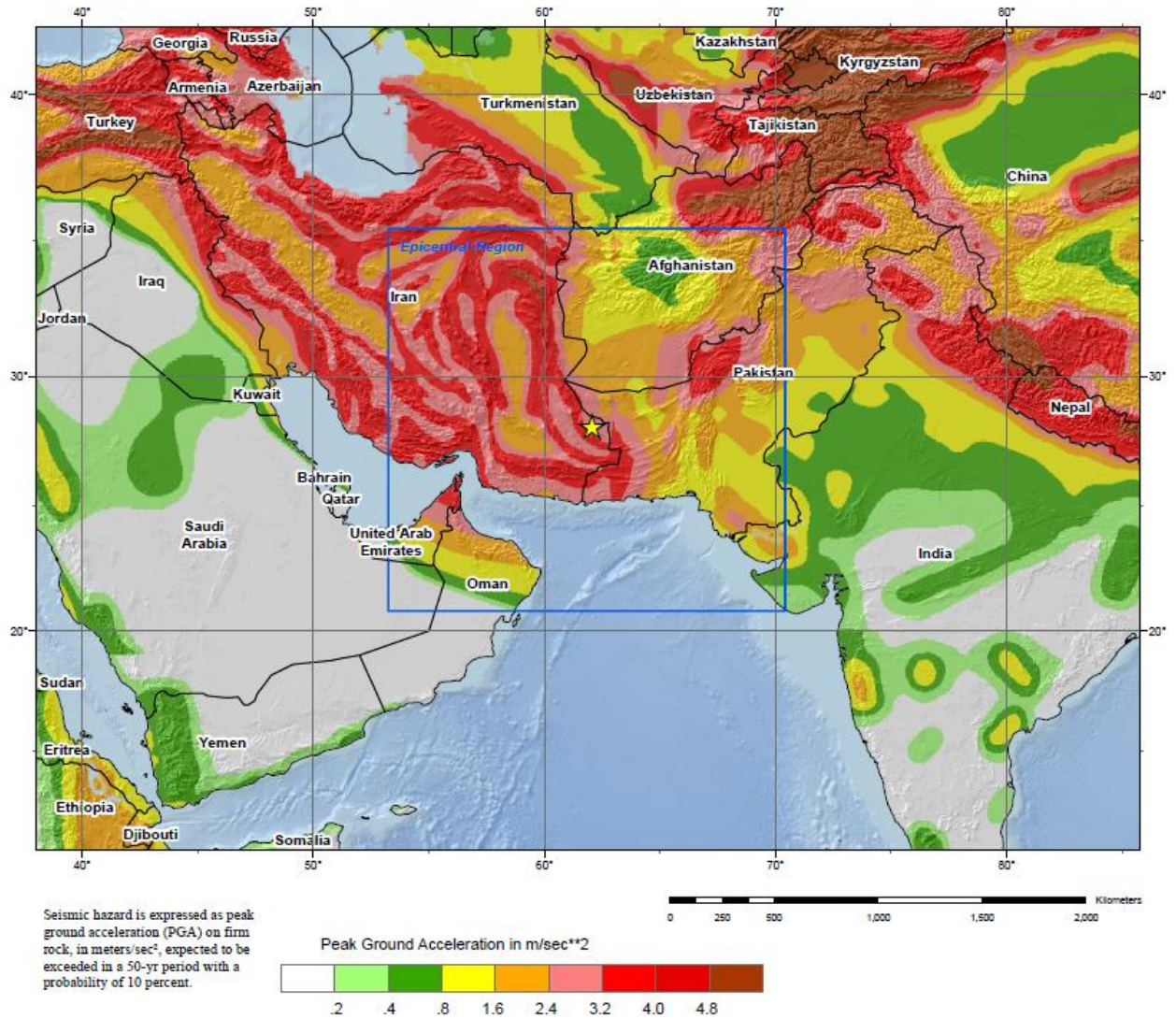


Figure 20 "Hazard map, Middle East, Peak ground acceleration expected in 50-years period with a probability of 10%"

Appendix F Eurocode Combination Loads

In this appendix chapter the characteristic, and design load values are summarised.

F.1 Characteristic Loads

From Table 48 to Table 53 the following nomenclature is used for the combination's loads.

- *D* Liquid Discharge (Self weight, Liquid filling, Wind)
- *I* Imposed load (Self Weight, Liquid filling, Imposed load, Wind)
- *S* Snow load (Self Weight, Liquid filling, Snow)
- *W_F* Wind and Full (Self Weight, Liquid filling full tank, Wind, Imposed load)
- *W_E* Wind and Empty (Self Weight, Liquid empty tank, Wind, Imposed load)
- *S_F* Seismic Full tank (Self Weight, Seismic Action, Liquid filling full tank, Imposed load)
- *S_E* Seismic Empty tank (Self Weight, Seismic Action, Liquid empty tank, Imposed load)

Table 48 "Eurocode Characteristic Load Combinations, case studied"

D	Self Weight [N]	1.52E+05	D	Self Weight [N]	1.36E+05
	Roof Load [N]	2.69E+04		Roof Load [N]	2.42E+04
	Surcharge Load [N]	3.00E+04		Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	3.19E+06		Liquid filling design filling [N]	3.19E+06
	Liquid filling full tank [N]			Liquid filling full tank [N]	
	Imposed Load [N]	5.30E+03		Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	1.78E+05		Total Vertical Load [N]	
	Wind load [N]	6.17E+04		Wind load [N]	6.17E+04
	Seismic load [N]			Seismic load [N]	
	Inner Ringwall soil [N]			Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	4.05E+03		Shell Weight per meter [N/m]	3.65E+03
	Foundation Concrete Load [N]			Foundation Concrete Load [N]	
Moment Wind [Nm]	3.14E+05	Moment Wind [Nm]	3.14E+05		
I	Self Weight [N]	1.52E+05	I	Self Weight [N]	1.36E+05
	Roof Load [N]	2.69E+04		Roof Load [N]	2.42E+04
	Surcharge Load [N]	3.00E+04		Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	3.19E+06		Liquid filling design filling [N]	3.19E+06
	Liquid filling full tank [N]			Liquid filling full tank [N]	
	Imposed Load [N]	5.30E+03		Imposed Load [N]	5.30E+03
	Total Vertical Load [N]			Total Vertical Load [N]	
	Wind load [N]	6.17E+04		Wind load [N]	6.17E+04
	Seismic load [N]			Seismic load [N]	
	Inner Ringwall soil [N]			Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	4.05E+03		Shell Weight per meter [N/m]	3.65E+03
	Foundation Concrete Load [N]			Foundation Concrete Load [N]	
Moment Wind [Nm]	3.14E+05	Moment Wind [Nm]	3.14E+05		

Table 49 "Eurocode Characteristic Load Combinations, case studied"

S	Self Weight [N]	1.52E+05
	Roof Load [N]	2.69E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	3.19E+06
	Liquid filling full tank [N]	
	Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	
	Wind load [N]	
	Seismic load [N]	
	Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	4.05E+03
	Foundation Concrete Load [N]	
Moment Wind [Nm]		
W_F	Self Weight [N]	1.52E+05
	Roof Load [N]	2.69E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	
	Liquid filling full tank [N]	3.99E+06
	Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	
	Wind load [N]	6.17E+04
	Seismic load [N]	
	Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	4.05E+03
	Foundation Concrete Load [N]	
Moment Wind [Nm]	3.14E+05	
W_E	Self Weight [N]	1.52E+05
	Roof Load [N]	2.69E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	0.00E+00
	Liquid filling full tank [N]	
	Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	
	Wind load [N]	6.17E+04
	Seismic load [N]	
	Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	4.05E+03
	Foundation Concrete Load [N]	
Moment Wind [Nm]	3.14E+05	

S	Self Weight [N]	1.36E+05
	Roof Load [N]	2.42E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	3.19E+06
	Liquid filling full tank [N]	
	Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	
	Wind load [N]	
	Seismic load [N]	
	Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	3.65E+03
	Foundation Concrete Load [N]	
Moment Wind [Nm]		
W_F	Self Weight [N]	1.36E+05
	Roof Load [N]	2.42E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	
	Liquid filling full tank [N]	3.99E+06
	Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	
	Wind load [N]	6.17E+04
	Seismic load [N]	
	Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	3.65E+03
	Foundation Concrete Load [N]	
Moment Wind [Nm]	3.14E+05	
W_E	Self Weight [N]	1.36E+05
	Roof Load [N]	2.42E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	8.04E+04
	Liquid filling full tank [N]	
	Imposed Load [N]	5.30E+03
	Total Vertical Load [N]	
	Wind load [N]	6.17E+04
	Seismic load [N]	
	Inner Ringwall soil [N]	
	Shell Weight per meter [N/m]	3.65E+03
	Foundation Concrete Load [N]	
Moment Wind [Nm]	3.14E+05	

Table 50 "Eurocode Characteristic Seismic Load Combinations, case studied"

SF	Self Weight [N]	1.52E+05
	Roof Load [N]	2.69E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	
	Liquid filling full tank [N]	3.19E+06
	Imposed Load [N]	2.27E+03
	Total Vertical Load [N]	
	Wind load [N]	
	Seismic load [N]	7.40E+05
	Inner Ringwall soil	
	Shell Weight per meter [N/m]	4.05E+03
	Foundation Concrete Load [N]	
	Moment Wind [Nm]	
	Moment Earthquake 1 [Nm]	2.96E+06
	Moment Earthquake 2 [Nm]	3.21E+06
Total Moment 1 [Nm]	2.96E+06	
Total Moment 2 [Nm]	3.21E+06	
SE	Self Weight [N]	1.52E+05
	Roof Load [N]	2.69E+04
	Surcharge Load [N]	3.00E+04
	Liquid filling design filling [N]	
	Liquid filling full tank [N]	
	Imposed Load [N]	2.27E+03
	Total Vertical Load [N]	
	Wind load [N]	
	Seismic load [N]	3.21E+04
	Inner Ringwall soil	
	Shell Weight per meter [N/m]	4.05E+03
	Foundation Concrete Load [N]	
	Moment Wind [Nm]	
	Moment Earthquake 1 [Nm]	1.96E+05
	Moment Earthquake 2 [Nm]	1.96E+05
Total Moment 1 [Nm]		
Total Moment 2 [Nm]		

F.2 Design Loads [Above Foundation]

Table 51 "Eurocode Design Load Combinations, case studied "

	APPROACH I		APPROACH II	APPROACH III
	Comb 1	Comb 2		
D				
Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05
Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04
Liquid filling design filling [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
Liquid filling full tank [N]				
Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
Total Vertical Load [N]	4.24E+06	4.17E+06	4.24E+06	4.24E+06
Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
Filling Rate [N]				
10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05
30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06
50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06
70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06
90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06
Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
Seismic load [N]				
Inner Ringwall soil pressure [kPa]				
Shell Weight per meter [N/m]	5.47E+03	4.05E+03	5.47E+03	5.47E+03
Foundation Concrete Load [N]				
Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
Eccentricity [m]	0.1111	0.10	0.11	0.11
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R
Effective Area [m^2]	48.49	48.70	48.49	48.49
I				
Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05
Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04
Liquid filling design filling [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
Liquid filling full tank [N]				
Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
Total Vertical Load [N]	4.24E+06	4.17E+06	4.24E+06	4.24E+06
Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
Filling Rate [N]				
10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05
30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06
50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06
70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06
90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06
Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
Seismic load [N]				
Inner Ringwall soil pressure [kPa]				
Shell Weight per meter [N/m]	5.47E+03	4.05E+03	5.47E+03	5.47E+03
Foundation Concrete Load [N]				
Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
Eccentricity [m]	0.1111	0.10	0.11	0.11
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R
Effective Area [m^2]	48.49	48.70	48.49	48.49
S				
Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05
Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04
Liquid filling design filling [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
Liquid filling full tank [N]				
Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
Total Vertical Load [N]	4.24E+06	4.17E+06	4.24E+06	4.24E+06
Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
Filling Rate [N]				
10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05
30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06
50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06
70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06
90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06
Wind load [N]				
Seismic load [N]				
Inner Ringwall soil pressure [kPa]				
Shell Weight per meter [N/m]	5.47E+03	4.05E+03	5.47E+03	5.47E+03
Foundation Concrete Load [N]				
Moment Wind [Nm]				
Eccentricity [m]	0.0000	0.00	0.00	0.00
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R
Effective Area [m^2]	50.27	50.27	50.27	50.27

	APPROACH I		APPROACH II	APPROACH III
	Comb 1	Comb 2		
D				
Self Weight [N]	1.84E+05	1.36E+05	1.84E+05	1.84E+05
Roof Load [N]	3.27E+04	2.42E+04	3.27E+04	3.27E+04
Liquid filling design filling [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
Liquid filling full tank [N]				
Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
Total Vertical Load [N]	4.21E+06	4.16E+06	4.21E+06	4.21E+06
Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
Filling Rate [N]				
10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05
30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06
50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06
70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06
90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06
Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
Seismic load [N]				
Inner Ringwall soil pressure [kPa]				
Shell Weight per meter [N/m]	4.93E+03	3.65E+03	4.93E+03	4.93E+03
Foundation Concrete Load [N]				
Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
Eccentricity [m]	0.1117	0.10	0.11	0.11
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R
Effective Area [m^2]	48.48	48.70	48.48	48.48
I				
Self Weight [N]	1.84E+05	1.36E+05	1.84E+05	1.84E+05
Roof Load [N]	3.27E+04	2.42E+04	3.27E+04	3.27E+04
Liquid filling design filling [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
Liquid filling full tank [N]				
Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
Total Vertical Load [N]	4.21E+06	4.16E+06	4.21E+06	4.21E+06
Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
Filling Rate [N]				
10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05
30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06
50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06
70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06
90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06
Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
Seismic load [N]				
Inner Ringwall soil pressure [kPa]				
Shell Weight per meter [N/m]	4.93E+03	3.65E+03	4.93E+03	4.93E+03
Foundation Concrete Load [N]				
Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
Eccentricity [m]	0.1117	0.10	0.11	0.11
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R
Effective Area [m^2]	48.48	48.70	48.48	48.48
S				
Self Weight [N]	1.84E+05	1.36E+05	1.84E+05	1.84E+05
Roof Load [N]	3.27E+04	2.42E+04	3.27E+04	3.27E+04
Liquid filling design filling [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
Liquid filling full tank [N]				
Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
Total Vertical Load [N]	4.21E+06	4.16E+06	4.21E+06	4.21E+06
Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
Filling Rate [N]				
10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05
30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06
50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06
70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06
90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06
Wind load [N]				
Seismic load [N]				
Inner Ringwall soil pressure [kPa]				
Shell Weight per meter [N/m]	4.93E+03	3.65E+03	4.93E+03	4.93E+03
Foundation Concrete Load [N]				
Moment Wind [Nm]				
Eccentricity [m]	0.0000	0.00	0.00	0.00
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R
Effective Area [m^2]	50.27	50.27	50.27	50.27

Table 52 "Eurocode Design Load Combinations, case studied "

W_F	Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05
	Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04
	Liquid filling design filling [N]				
	Liquid filling full tank [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
	Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
	Total Vertical Load [N]	4.24E+06	4.17E+06	4.24E+06	4.24E+06
	Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
	Filling Rate [N]				
	10%	5.98E+05	5.18E+05	5.98E+05	5.98E+05
	30%	1.79E+06	1.56E+06	1.79E+06	1.79E+06
	50%	2.99E+06	2.59E+06	2.99E+06	2.99E+06
	70%	3.99E+06	3.63E+06	3.99E+06	3.99E+06
	90%	3.99E+06	3.99E+06	3.99E+06	3.99E+06
	Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
	Seismic load [N]				
	Inner Ringwall soil pressure [kPa]				
	Shell Weight per meter [N/m]	5.47E+03	4.05E+03	5.47E+03	5.47E+03
	Foundation Concrete Load [N]				
	Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
	Eccentricity [m]	0.1111	0.10	0.11	0.11
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	
Effective Area [m^2]	48.49	48.70	48.49	48.49	
W_E	Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05
	Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04
	Liquid filling design filling [N]				
	Liquid filling full tank [N]				
	Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
	Total Vertical Load [N]	2.49E+05	1.85E+05	2.49E+05	2.49E+05
	Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
	Filling Rate [N]				
	10%				
	30%				
	50%				
	70%				
	90%				
	Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
	Seismic load [N]				
	Inner Ringwall soil pressure [kPa]				
	Shell Weight per meter [N/m]	5.47E+03	4.05E+03	5.47E+03	5.47E+03
	Foundation Concrete Load [N]				
	Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
	Eccentricity [m]	1.8918	2.20	1.89	1.89
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	
Effective Area [m^2]	21.17	16.91	21.17	21.17	

W_F	Self Weight [N]	1.84E+05	1.36E+05	1.84E+05	1.84E+05
	Roof Load [N]	3.27E+04	2.42E+04	3.27E+04	3.27E+04
	Liquid filling design filling [N]				
	Liquid filling full tank [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06
	Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
	Total Vertical Load [N]	4.21E+06	4.16E+06	4.21E+06	4.21E+06
	Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
	Filling Rate [N]				
	10%	5.98E+05	5.18E+05	5.98E+05	5.98E+05
	30%	1.79E+06	1.56E+06	1.79E+06	1.79E+06
	50%	2.99E+06	2.59E+06	2.99E+06	2.99E+06
	70%	3.99E+06	3.63E+06	3.99E+06	3.99E+06
	90%	3.99E+06	3.99E+06	3.99E+06	3.99E+06
	Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
	Seismic load [N]				
	Inner Ringwall soil pressure [kPa]				
	Shell Weight per meter [N/m]	4.93E+03	3.65E+03	4.93E+03	4.93E+03
	Foundation Concrete Load [N]				
	Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
	Eccentricity [m]	0.1117	0.10	0.11	0.11
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	
Effective Area [m^2]	48.48	48.70	48.48	48.48	
W_E	Self Weight [N]	1.84E+05	1.36E+05	1.84E+05	1.84E+05
	Roof Load [N]	3.27E+04	2.42E+04	3.27E+04	3.27E+04
	Liquid filling design filling [N]				
	Liquid filling full tank [N]				
	Imposed Load [N]	7.96E+03	6.90E+03	7.96E+03	7.96E+03
	Total Vertical Load [N]	2.25E+05	1.70E+05	2.25E+05	2.25E+05
	Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04
	Filling Rate [N]				
	10%				
	30%				
	50%				
	70%				
	90%				
	Wind load [N]	9.25E+04	8.02E+04	9.25E+04	9.25E+04
	Seismic load [N]				
	Inner Ringwall soil pressure [kPa]				
	Shell Weight per meter [N/m]	4.93E+03	3.65E+03	4.93E+03	4.93E+03
	Foundation Concrete Load [N]				
	Moment Wind [Nm]	4.71E+05	4.08E+05	4.71E+05	4.71E+05
	Eccentricity [m]	2.0946	2.40	2.09	2.09
	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	Verified because e<0.6R	
Effective Area [m^2]	18.35	14.33	18.35	18.35	

Table 53 "Eurocode Design Seismic Load Combinations, case studied "

SF	Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05	
	Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04	
	Liquid filling design filling [N]					
	Liquid filling full tank [N]	3.99E+06	3.99E+06	3.99E+06	3.99E+06	
	Imposed Load [N]	3.41E+03	2.96E+03	3.41E+03	3.41E+03	
	Total Vertical Load [N]	4.23E+06	4.17E+06	4.23E+06	4.23E+06	
	Surcharge Load [N]	3.00E+04	3.00E+04	3.00E+04	3.00E+04	
	Filling Rate [N]					
	10%	4.78E+05	4.15E+05	4.78E+05	4.78E+05	
	30%	1.44E+06	1.24E+06	1.44E+06	1.44E+06	
	50%	2.39E+06	2.07E+06	2.39E+06	2.39E+06	
	70%	3.35E+06	2.90E+06	3.35E+06	3.35E+06	
	90%	3.99E+06	3.73E+06	3.99E+06	3.99E+06	
	Wind load [N]					
	Seismic load [N]	1.11E+06	9.62E+05	1.11E+06	1.11E+06	
	Inner Ringwall soil [N]					
	Shell Weight per meter [N/m]	5.47E+03	4.05E+03	5.47E+03	5.47E+03	
	Foundation Concrete Load [N]					
	Moment Wind [Nm]					
	Moment Earthquake 1 [Nm]	4.43E+06	3.84E+06	4.43E+06	4.43E+06	
	Moment Earthquake 2 [Nm]	4.81E+06	4.17E+06	4.81E+06	4.81E+06	
	Total Moment 1 [Nm]	4.43E+06	3.84E+06	4.43E+06	4.43E+06	
	Total Moment 2 [Nm]	4.81E+06	4.17E+06	4.81E+06	4.81E+06	
	Eccentricity 1 [m]	1.05	0.92	1.05	1.05	
		because	because	Verified because	Verified because	
	Eccentricity 2 [m]	1.14	1.00	1.14	1.14	
		Verified	Verified	Verified because	Verified because	
	Effective Area 1 [m^2]	33.70	35.66	33.70	33.70	
	Effective Area 2 [m^2]	32.34	34.45	32.34	32.34	
	SE	Self Weight [N]	2.05E+05	1.52E+05	2.05E+05	2.05E+05
		Roof Load [N]	3.63E+04	2.69E+04	3.63E+04	3.63E+04
		Liquid filling design filling [N]				
		Liquid filling full tank [N]				
Imposed Load [N]		3.41E+03	2.96E+03	3.41E+03	3.41E+03	
Total Vertical Load [N]		2.44E+05	1.81E+05	2.44E+05	2.44E+05	
Surcharge Load [N]		3.00E+04	3.00E+04	3.00E+04	3.00E+04	
Filling Rate [N]						
10%						
30%						
50%						
70%						
90%						
Wind load [N]						
Seismic load [N]		4.81E+04	4.17E+04	4.81E+04	4.81E+04	
Inner Ringwall soil [N]						
Shell Weight per meter [N/m]		5.47E+03	4.05E+03	5.47E+03	5.47E+03	
Foundation Concrete Load [N]						
Moment Wind [Nm]						
Moment Earthquake 1 [Nm]		2.94E+05	2.55E+05	2.94E+05	2.94E+05	
Moment Earthquake 2 [Nm]		2.94E+05	2.55E+05	2.94E+05	2.94E+05	
Total Moment 1 [Nm]						
Total Moment 2 [Nm]						
Eccentricity 1 [m]		1.20	1.41	1.20	1.20	
		because	because	Verified because	Verified because	
Eccentricity 2 [m]		1.20	1.41	1.20	1.20	
		Verified	Verified	Verified because	Verified because	
Effective Area 1 [m^2]		31.28	28.24	31.28	31.28	
Effective Area 2 [m^2]		31.28	28.24	31.28	31.28	

Appendix G API 650 Combination Loads

Table 54 "API 650 Load Combinations, case studied "

1	Fluid and internal Pressure	Dead Load	195.78	kN
		Fluid Load	3189.75	kN
3	Wind and internal Pressure	Dead Load	195.78	kN
		Wind Lateral Load	123.35	kN
		Moment	616.75	kNm
		Eccentricity	3.15	m
5	a) Gravity Loads	Dead Load	195.78	kN
		Roof live load	50.27	kN
	b) Gravity Loads	Dead Load	195.78	kN
		Roof live load	20.11	kN
6	Seismic	Dead Load	195.78	kN
		Roof Load	26.90	
		Fluid Load	3189.75	kN
		Horizontal Load	625.65	kN
		Vertical Seismic Force [kN]	349.13	
		Moment	2316.90	kNm
		Eccentricity	0.68	m

- 1 Fluid and internal pressure (Self weight, Liquid Filling)
- 3 Wind and Internal pressure (Self weight, Wind)
- 5a) Gravity Load (Self Weight, Imposed load)
- 5b) Gravity Load (Shell Load, 0.4 Roof Load)
- S Seismic Full tank (Self Weight, Seismic Action, Liquid filling full tank, Imposed load)

Appendix H Load Combinations ACI 318

Table 55 "ACI 318 Load Combinations, case studied"

1	D ₀ [kN]	3386	5	W _s +W _r +W _b [kN]	196
	Total Vertical Load [kN]	4063		W _p [kN]	3190
	Twisting Moment (W _s +W _r +W _b) [kNm]	37		Live load [kN]	50
	Twisting Moment (W _p) [kNm]	781		Total Vertical Load [kN]	4123
	Total Twisting moment [kN]	819		Twisting Moment (W _s +W _r +W _b) [kNm]	37
2	W _t [kN]	3945	6	Twisting Moment (W _p) [kNm]	781
	Total Vertical Load [kN]	4734		Twisting Moment Live load [kNm]	10
	Twisting Moment (W _p) [kNm]	966		Total Twisting moment [kNm]	828
	Total Twisting moment [kNm]	966		W _s +W _r +W _b [kN]	196
3	W _s +W _r +W _b [kN]	196	8	W _p [kN]	3190
	W _p [kN]	3190		Live load [kN]	20
	Total Vertical Load [kN]	4063		Total Vertical Load [kN]	4087
	Twisting Moment (W _s +W _r +W _b) [kNm]	37		Twisting Moment (W _s +W _r +W _b) [kNm]	37
	Twisting Moment (W _p) [kNm]	781		Twisting Moment (W _p) [kNm]	781
	Wind moment [kNm]	617		Twisting Moment Live load [kNm]	4
	Vertical Wind compressive force [kN]	207		Total Twisting moment [kNm]	823
	Twisting Wind Moment [kNm]	372		W _s +W _r +W _b [kN]	196
	Total Twisting moment [kNm]	1191		W _p [kN] (vertical acceleration)	166
	4	W _s +W _r +W _b [kN]		196	8
W _p [kN]		3190	Total Vertical Load [kN]	527	
Total Vertical Load [kN]		4063	Twisting Moment (W _s +W _r +W _b) [kNm]	37	
Twisting Moment (W _s +W _r +W _b) [kNm]		37	Twisting Moment (W _p) [kNm]	41	
Twisting Moment (W _p) [kNm]		781	Twisting Moment Live load [kNm]	10	
Wind moment [kNm]		617	Total Twisting moment [kNm]	87	
Vertical Wind compressive force [kN]		1958	Where:		
Twisting Wind Moment [kNm]		372	Shell Weight	W _s	
Total Twisting moment [kNm]		1191	Roof Weight	W _r	
Longitudinal Moment [kNm]		4989	Base Plate Weight	W _b	
		Test Medium Weight	W _t		
		Product Weight	W _p		

Table 56 "Combination Loads in according to ACI 318"

Loading Combinations – Allowable Stress Design (Service Load)		
Load Comb	Load Combination	Description
1	$D_s + D_0$	Operating Weight
2	$D_s + Dt + Pt$	Test Weight + Test Pressure
3	$D_s + (De \text{ or } D_0) + W$	Empty or Operating Weight + Wind
4	$D_s + (De \text{ or } D_0) + W$	Empty or Operating Weight + Wind
5	$D_s + D_0 + (L \text{ or } S)$	Operating Weight + Live
6	$D_s + (De \text{ or } D_0) + 0.4(L \text{ or } S)$	Empty or Operating Weight + Live
7	$D_s + D_0 + 0.1 S + E_0 + 0.4P_i$	Operating Weight + Snow + Earthquake + Internal Pressure
8	$D_s + D_0 + 0.1 S + E_0$	Operating Weight + Snow + Earthquake

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